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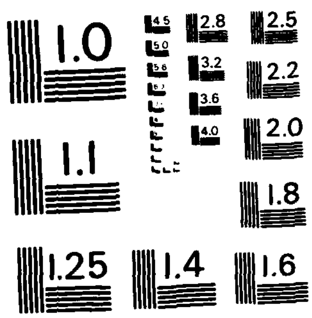

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EMBANKMENT CRITERIA AND PERFORMANCE REPORT

6

MARCH 1983

AD-A132 763

MISSOURI RIVER

FORT RANDALL DAM-
LAKE FRANCIS CASE

US Army Corps of Engineers
Omaha District

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Instrumentation of Dams	Fort Randall Dam

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

Foundation features, earthfill design, construction, instrumentation, and performance history of the Fort Randall Dam embankment are summarized. The 165-foot high embankment was constructed in six earthwork stages, from 1947 through 1955, and is composed of approx. 50,000,000 cubic yards of rolled earthfill and chalkfill materials. It is founded on pervious alluvial soils in the valley and loess and glacial till in the abutments. Relief wells aid in controlling seepage. Instrumentation includes piezometers, settlement gages, crest and slope movement markers, tiltmeters, and strong motion accelerographs.

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MISSOURI RIVER
FORT RANDALL DAM - LAKE FRANCIS CASE
SOUTH DAKOTA

EMBANKMENT CRITERIA AND PERFORMANCE REPORT

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PERTINENT DATA

1. EMBANKMENT

Type	Rolled Earth and Chalk Fill
Height Above Stream Bed	165 Feet
Height Above Flood Plain	140 Feet
Length	10,700 Feet
Crest Elevation	1,395 Feet, m.s.l.
Crest Width	60 Feet
Volume	50,000,000 Cubic Yards
Closure Date	20 July 1952

2. SPILLWAY

Type	Concrete-lined Chute w/Gated Weir
Width	1,000 Feet
Weir Crest Elevation	1,346 Feet, m.s.l.
Gates, Type	Radial Tainter
Gates, Number and Size	21-40 Feet by 29 Feet
Elevation, Top of Gates	1,375 Feet, m.s.l.
Design Discharge Capacity at Elevation 1379.2	629,000 c.f.s.

3. OUTLET WORKS

Type	12 Concrete-lined Tunnels With Control In Intake Structure
Number, Diameter, and Length of Flood Control Tunnels	Four, 22-Foot Diameter, 873 Feet Long
Number, Diameter, and Length of Power Tunnels	Eight, 22-Foot Diameter, 873 Feet Long
Type, Number, and Size of Intake Gates	Bulkhead, 24 - 11 Feet by 23 Feet
Invert Elevation of Intake	1,229 Feet, m.s.l.
Discharge Capacity of Flood Control Tunnels	128,000 c.f.s.

4. POWERHOUSE

Length	561 Feet
Width	78 Feet
Number of Generating Units	8

Generating Capacity, Each Unit	42,105 kVA
Total Installed Capacity	320,000 Kilowatts
Power on Line	March, 1954

5. **RESERVOIR**

Drainage Area Above Dam	263,480 Square Miles
Drainage Area, Fort Randall Dam to Big Bend Dam	14,150 Square Miles
Storage Capacity at Maximum Pool (Elev. 1375)	5,600,000 Acre-Feet
Storage Capacity at Maximum Normal Operating Pool (Elev. 1365)	4,620,000 Acre-Feet
Flood Control Reserve (Elev. 1365 to Elev. 1375)	980,000 Acre-Feet
Annual Flood Control and Multiple-Purpose (Elev. 1350 to Elev. 1365)	1,300,000 Acre-Feet
Carry-Over Multiple-Purpose (Elev. 1320 to Elev. 1350)	1,730,000 Acre-Feet
Dead Storage, Elev. 1320	1,600,000 Acre-Feet
Length of Pool at Elev. 1365	107 Miles
Maximum Normal Operating Pool Elevation	1,365 Feet, m.s.l.
Base, Seasonal Flood Control Pool	1,350 Feet, m.s.l.

MISSOURI RIVER
FORT RANDALL DAM - LAKE FRANCIS CASE
SOUTH DAKOTA

EMBANKMENT CRITERIA AND PERFORMANCE REPORT

1. INTRODUCTION.

1.1 **Purpose and Scope of Report.** This report provides a summary record of significant design, construction, and operational data on the Fort Randall Dam embankment. It was prepared in accordance with ER 1110-2-1901, "Embankment Criteria and Performance Report," dated 31 December 1981 and is for use by engineers to familiarize themselves with the project, reevaluate the embankment when needed, and for guidance in designing comparable future projects.

The report presents a general description of the foundation conditions, the type of material and placement methods of the various sections of the embankment, the design considerations on stability and seepage control, instrumentation, significant operational events, and an evaluation of the condition of the embankment. Pertinent drawings, design and construction data, and photos are included. A more detailed description of the foundation conditions is contained in the Construction Foundation Report prepared in 1980.

1.2 **Brief Description and Purpose of Project.** The Fort Randall Dam-Lake Francis Case Project is one of six multipurpose dam projects on the Missouri River for flood control, irrigation, navigation, power, and recreational purposes. The project is operated and maintained by the U.S. Army Corps of Engineers, Omaha District. The dam is located about 105 river miles downstream of Big Bend Dam and about 70 river miles upstream of Gavins Point Dam. As shown on the location map on Plate A-1, it is approximately 6 miles south of Lake Andes, South Dakota, and is 880 river miles (1960 adjustment) above the mouth of the Missouri River.

The project consists of an earth embankment, a multigated concrete spillway, an intake structure, multitunneled outlet works, and a hydroelectric power generating plant. The embankment is about 165 feet high above

the streambed and extends about 10,700 feet from the right abutment to the spillway structure located in the left abutment. A plan and typical sections of the project structures are shown on Plate A-2. Photo No. 1 shows an aerial view of the project.

1.3 **Authorization of Dam Project.** The Fort Randall Dam and Reservoir project was authorized by the Flood Control Act, approved 22 December 1944 (Public Law 534, 78th Congress, 2nd Session).

1.4 **Design and Construction of Project.** The project was designed by the U.S. Army Corps of Engineers, Omaha District. Members of the Board of Consultants were Dr. Arthur Casagrande, Mr. L. F. Harza, Dr. L. C. Glenn, Mr. J. D. Justin, Mr. S. O. Harper, and Mr. W. H. McAlpine.

The embankment was constructed in six earthwork contract stages as listed below.

Embankment Construction Stage	Contract No.	Contractor	Date Started
Foundation Preparation Left Bank Chute	W-25-066-ENG-1350	Peter Kiewit Sons Co.	Sep 47
Initial Earthwork	W-25-066-ENG-1434	Western Contracting Corp.	Feb 48
Earthwork Stage II	W-25-066-ENG-1837	Western Contracting Corp.	Feb 49
Earthwork Stage III	DA-25-066-ENG-363	Western Contracting Corp.	Jun 50
Earthwork Stage IV	DA-25-066-ENG-2139	Western Contracting Corp.	Nov 52
Earthwork Stage V	DA-25-066-ENG-2531	List and Clark Construction Co.	Jul 53

Work on the outlet works tunnel started in December 1948 and on the spillway, in April 1951. The river closure and diversion was made on 20 July 1952 during earthwork Stage III and the embankment was essentially completed by June 1955. The embankment pressure relief well system was installed by Bassett Drilling Company during 1953 and 1954 under Contract No. DA-25-066-ENG-2571. Some of the wells were extended by the Stage V earthwork contractor. All of the contracts were administered by the Corps of Engineers, Omaha District. Field supervision was by personnel of the Fort Randall area office which was located in Pickstown, a town constructed to accommodate the large number of people who were involved in the construction of the dam project.

1.5 **Significant Operational Events.** The reservoir was initially raised to Elev. 1,340 in 1954 and to Elev. 1,350 in 1957. Since 1971, it has fluctuated annually from a low of about Elev. 1,340 in the winter to about Elev. 1,355 to Elev. 1,360 in the summer. It attained its highest level of Elev. 1,366.5 on 23 June 1967.

Erosion of the upstream chalk berm by wave action since 1957 required the placement of riprap protection in 1981 along the erosion scarp which extended over the full length of the berm.

1.6 **Reference Project Publications.** Detailed information on the constructed dam foundation, evaluation of relief wells, project maintenance, and periodic inspections are included in the following Omaha district manual and reports:

a. Construction Foundation Report (September 1980).
b. Embankment Relief Well Study (August 1982).
c. Operation and Maintenance Manual (1982).
d. Periodic Inspection Reports Nos. 1, 2, 3, and 4, the latest of which is dated June 1981.

2. GEOLOGY.

2.1 **General.** Fort Randall Dam is located in the eastward sloping Missouri Plateau Section of the Great Plains physiographic province. The dam lies across an entrenched glacial melt-water channel at the southwestern margin of glaciated eastern South Dakota. During the mid-Pleistocene Period melt water ran along the front of the Illinoian Glacier which had advanced into South Dakota. When the glacier retreated the melt-water channel had entrenched enough to remain as the infant Missouri River. The river trench was partly filled during the later Wisconsin glaciation, but it retrenched its valley in the Recent Period.

The dam is located on generally flat lying, well compacted, sedimentary formations deposited by continental seas during the Cretaceous Period. Although the formations may be locally considered horizontal, they display an undulating trend on a larger scale. This is due to minor folding or differential compaction over the underlying Precambrian basement structure. The Precambrian basement consists predominantly of a NW-SE trending quartzite ridge in eastern South Dakota giving way in east-central South Dakota to a broad, flat structural platform which slopes gently westward.

The valley at the damsite is approximately 8,000 feet wide with a low flood plain elevation of about 1,250. The left abutment is formed by a promontory which rises between two deep tributary ravines to 160 feet above the flood plain; the ground then rises gently for a distance of about 3,000 feet to an elevation of 1,440. The left flood plain is 250 feet wide at the dam axis and increases in width upstream to about 2,000 feet at the head of the spillway approach channel. The right flood plain was just a few feet wide on the dam axis, but increased rapidly in width downstream. The right abutment is on a 60-foot high promontory formed by an eroded river terrace. The terrace rises to the west with a 1V on 40H slope for a distance of about 2,600 feet; it then rises on a 1V on 20H slope for about 1,400 more feet. Prior to construction of the dam, the river at the site was divided by an

alluvial island into a left 1,100-foot-wide channel and a right 1,500-foot-wide channel. The island was about 1-1/4 mile long, 1,400 feet wide, and had a surface elevation of about 1,245 feet.

2.2 **Subsurface Explorations.** After the present damsite was selected, 11 rotary and churn drill holes were made in 1941. An additional 51 borings and three test pits were made in 1942. From 1946 and throughout the design and construction period, over 450 additional borings and 28 test pits were made to determine the foundation conditions. The borings included 3-inch core borings, 6-inch core borings, 12-inch auger holes, and three 36-inch diameter calyx holes. Boring locations are shown on Plate A-8. The drilling program and other foundation investigations such as bedrock contouring, pumping tests to determine the permeability of the valley sands, and geologic mapping of excavations during construction are described in the referenced Construction Foundation Report.

2.3 **Ground Water.** Ground water levels were recorded in numerous exploratory drill holes prior to construction. The water table in the river valley and was as much as 12.0 feet below ground surface, but it only varied in elevation from about 1,237 to 1,240, the river surface elevation. In the left abutment, the water table ranged in depth from about 73 to 142 feet at elevations between 1,304 to 1,360 with an average of about 1,318. The water table was found in both overburden and bedrock, but for the most part, it was in or near the overburden-bedrock contact. In the right abutment, the water table generally occurred in overburden between 15 to 75 feet below the ground surface, and the elevations varied from 1,237 to 1,288 with an average of about 1,240.

A number of artesian springs and wells were found in the Missouri River Valley upstream and downstream from the Fort Randall damsite. One of these springs was located at about Sta. 80+00 near the axis of the embankment. The spring water was tested and found to originate from the Dakota sandstone at about Elev. 750. Apparently, the river had eroded enough of the overlying sediments to allow the artesian pressured Dakota water to force its way to

the surface through fracture zones. The presence of this and similar springs in the reservoir was not considered a critical problem.

2.4 **Overburden.** The overburden at the damsite consists predominantly of fluvial sands in the river valley, clays and silts in the river terrace of the right abutment, loess overlying the river terrace, and glacial till in the upper left abutment. A geologic profile along the axis of the embankment is shown on Plate A-6.

2.4.1 **Fluvial Valley Sands.** The upper portion of the valley fill consists of alluvial sand which ranges from fine to medium in grain size. The fine-to-medium sizes predominate throughout this fill, but with increasing depth the sand becomes more coarse and in part, gravelly. The thickness of the alluvium varied across the river valley from a relatively shallow depth adjacent to the abutments to a maximum thickness of 175 feet in the vicinity of the midstream island. Some thin layers of gravel occurred in this material, but they appeared to be restricted lenses with no great lateral extent. There were also occasional small seams of clay, silty sand, lime, and lignite float. The lower deposits are probably of glacial origin.

Gradation curves for the sand are shown on Plate A-24. Well pumping tests indicated a permeability from 0.001 to 0.004 ft/sec in the upper medium-grained sands; however, the permeability for the deep sands containing more gravel was found to be about 0.001 ft/sec (86.4 ft./day). This suggested that the gravelly sand contained fines which reduced its permeability. The coefficient of permeability for remolded samples of the sand ranged between 3 and 10 feet per day. The coefficients of internal friction for remolded samples of the sand varied between 0.6 and 0.7.

2.4.2 **River Terrace Clays.** An extensive river terrace formed the right abutment of the dam. It consisted of poorly compacted alluvial clays and silts more than 100 feet in thickness. The material was classified as a lean to fat clay with approximately 77 to 98 percent finer than the No. 200 sieve size as shown by the gradation curves on Plate A-26. The in-place

dry density of the clay averaged about 90 pounds per cubic foot (pcf) and the moisture content varied between 20 and 35 percent. The consistency of the material ranged from friable to stiff above the ground water table, but varied from stiff to soft at greater depths. The Atterberg Limits for the material varied widely, with liquid limits varying from 32 to 90 percent and plasticity indexes ranging from 10 to 58 percent. Direct shear and triaxial compression test results indicated a design shear strength of tan ϕ = 0.22 and cohesion = 0.4 tons per square foot. Sample data sheets and summaries of the shear tests are presented on Plates A-28 through A-32.

2.4.3 **Loess.** A blanket of loess covered the right abutment terrace clay deposit. It varied in thickness from about 50 feet at the river bluff to just a few feet in the upper gently sloping terrace area. The upper several feet of weathered zone did not have the low moisture content which is normally characteristic of loess, and it was somewhat denser than the unweathered material. In its natural state the loess has low density, ranging between 75 and 85 pounds per cubic foot, with up to 50 percent voids. It has a liquid limit of about 33 percent and a plasticity index of about 10. Its natural moisture content is about 10 to 15 percent. A shear strength of tan ϕ = 0.65 and cohesion = 0 was determined by direct shear tests and was selected for design. The material has a uniform grain size which is restricted almost entirely to silt and fine sand sizes as shown in the gradation curves on Plate A-25. The silt and sand grains are characteristically bonded by interstitial clay particles which give dry loess the strength to stand in vertical bluffs.

2.4.4 **Glacial Till.** Glacial till formed the overburden in the left abutment with a maximum thickness of 90 feet in the embankment foundation area. It consisted of a heterogeneous mixture of clay, silt, sand, gravel, and boulders with some fragmented shale and chalk. The range in gradation is shown on Plate A-27. The material is predominately a sandy clay, with an average liquid limit of about 40 to 45 and average plasticity index of about 20 to 25. Tests on 18 undisturbed samples indicated that the in-place dry density ranged from about 90 to 114 pounds per cubic foot (pcf)

and averaged about 99 pcf. Tests on a few of the larger samples, however, showed an average density of about 102 pcf. Moisture content averaged about 20 percent. Consolidation tests on the undisturbed glacial drift material, in which the specimen was loaded to its natural overburden load and then immersed, did not show any tendency of the material to slump or rapidly consolidate.

2.5 **Bedrock.** The rock formations at Fort Randall Dam are well indurated, compact sedimentary marine deposits of the upper Cretaceous Period. The stratigraphic sequence in ascending order is Greenhorn limestone, Carlile shale, Niobrara chalk, and Pierre shale. Plate A-6 shows a geologic column and geologic profile along the dam axis. Carlile shale and the Niobrara Formation constitute most of the bedrock foundation at the dam site. Formations at the dam site are flat-lying with a gentle dip of a few feet per mile to the northwest. A bedrock contour map is presented on Plate A-7. Bedding planes vary from being inches apart to as much as 5 feet apart in some portions of the Niobrara Formation. They often contain thin layers of gypsum, calcite, or clay. Nearly vertical, randomly oriented joints are common in rock exposures, but less common at depth. There is occasional faulting throughout the area, but usually with no more than a few feet of displacement and relatively short lateral extent. The faulting at the damsite was comparatively minor, but not uncomplicated. Both jointing and faulting in the chalky Niobrara Formation were relatively tight and prohibitive to solution; no caves, caverns, or solution channels were found in the area.

2.5.1 **Greenhorn Limestone.** This formation is generally recognized as a hard fossiliferous limestone. The predominant constituent is crystalline calcium carbonate, but it also has a considerable amount of clay and fine sand. Drill holes in the right abutment penetrated about 45 feet of Greenhorn limestone below Elev. 910.

2.5.2 **Carlile Shale.** The Carlile shale is mostly sandy to clayey shale with some interbedded sandstone. It is about 265 feet thick,

but the upper 100$\pm$ feet beneath the valley had eroded and filled with alluvium. It is in the river valley that the Carlile shale forms the flooring bedrock and creates the most concern as a foundation material. There is a relatively weak contact plane between the eroded Carlile shale and the overlying valley sands. Consolidated direct shear tests on samples of the shale indicated a design shear strength of tan ϕ = 0.3 and cohesion = 0.1 ton per square foot. The top of the formation under the left abutment and the right terrace is at about Elev. 1,175, approximately 65 feet below the river level.

2.5.3 **Niobrara Formation.** The Niobrara Formation is the uppermost formation of the Colorado Group, and is the oldest exposed bedrock in the area. It is a dark gray, argillaceous, soft but firm chalk and chalky shale which contains many microscopic shells of Foraminifera and Ostracoda. The color changes to a buff or light gray when the formation is weathered. The chalk is a porous rock, but the voids are so poorly interconnected that the rock is relatively impervious. It is a massive, coherent rock, and although it is soft enough to be scratched with a fingernail, it is also tough and resilient enough to resist fracturing. Fresh exposures of the chalk withstand repeated cycles of wetting and drying without appreciable deterioration. Thin layers of bentonitic clay with thicknesses up to 2 inches occur throughout the formation, but they are more concentrated in the upper 20 feet. Tests have revealed that after being thoroughly dried these clays, unlike pure bentonite, will not swell greatly when saturated.

The Niobrara Formation is the predominant sedimentary material at the damsite. It extends for about 145 feet in thickness from Elev. 1,175 to Elev. 1,320; this is about 65 feet below the river level to an average height of 80 feet above the river level. It is the most stable bedrock in the Fort Randall area and is the foundation for the outlet works, the spillway, and the powerplant. It was also the major rock encountered in construction excavations. Dry unit weight of the chalk varied from 85 to 114 pcf, and moisture content ranged from 20 to 30 percent.

2.5.4 **Pierre Shale**. The Pierre shale is the uppermost formation at the damsite, and its contact with the underlying Niobrara Formation occurs sharply at about Elev. 1,320. The formation is susceptible to landsliding, and it may be generally described as a noncalcareous to highly calcareous, gray, green, brown, or black, tough, gummy, marine shale with zones of bentonite seams and iron-manganese concretions. It is commonly divided into eight members; however, erosion has removed all but the lower member and therefore, the shale had only a minor influence on the dam construction. Excavation for the spillway encountered about 30 feet of Pierre shale, but no part of the structure was founded on the formation. A thin remnant of the shale, however, formed the bedrock beneath a small portion of the dam where the embankment is relatively low.

3. **EMBANKMENT SECTION.** A typical section of the embankment is shown on Plate A-2 and sections at various locations along the embankment are shown on Plates A-4 and A-5. The embankment has a maximum height of about 165 feet above the river bed and has an average height of about 140 feet over the flood plain. The crest of the embankment is 60 feet wide and is at Elev. 1,395 feet, mean sea level. The embankment section consists mainly of a central wide-based compacted impervious earth fill section and dumped chalk fill outer berm sections. An upstream impervious fill blanket adjacent to the central impervious section reduces uplift pressures beneath the embankment by lengthening the seepage path. In addition to the stability provided by the downstream chalk berm, the relief wells along the toe of the central impervious section provide assurance against the development of excessive underseepage pressures. The flat 1V on 15H surface of the upstream chalk berm was considered an acceptable alternative to the more expensive conventional riprap protection. Except for the areas inundated by the reservoir and the riprapped slope in the vicinity of the outlet works intake structure, the entire embankment is grassed for protection against surface erosion.

4. **CONSTRUCTION STAGES.** The embankment was constructed in six earthwork stages under separate contracts. The river was diverted through the power

and outlet works structures and tunnels during Earthwork Stage III. Diversion of the river, therefore, controlled the extent and sequence of construction for the various earthwork stages. Work covered under each stage is summarized below.

4.1 **Foundation Preparation - Left Bank Chute. (September, 1947 - January, 1948).** This preparatory work was required to provide a dry foundation for the embankment from the left bank of the river to the toe of the left abutment. It was completed during the fall and early winter months in preparation for the initial embankment earthwork stage that was scheduled to start the following spring. Approximately 390,000 cubic yards of compacted fill up to a maximum thickness of about 10 feet, to about Elev. 1,252 were placed under this contract.

4.2 **Initial Earthwork (February 1948 - December 1948).** Work under this stage included partial embankment construction from the river to the left abutment. It also included excavation in the upstream and downstream portal areas of the power and outlet works tunnels and partial excavation in the spillway approach channel. Two test embankments, one of shale and the other of excavated chalk material, were constructed during this phase and are discussed in subparagraph 6.5, Chalk Berm Fill. Approximately 6,725,000 cubic yards of compacted earth fill were placed to elevations varying from Elev. 1,271.5 over the left bank to Elev. 1,365 over the left abutment. The fill material was predominately glacial drift overburden excavated from the outlet works and spillway areas. Approximately 4,510,000 cubic yards of chalk and shale were also excavated from these areas and were used primarily in the construction of the upstream and downstream embankment chalk berms.

4.3 **Earthwork Stage II (February 1949 - October 1949).** Work under this contract was entirely on the left side of the river and was essentially a continuation of the activities performed under the previous earthwork contract. Approximately 4,380,000 cubic yards of compacted earthfill and 147,000 cubic yards of pervious channel fill were placed. The compacted

embankment over the left bank varied up to Elev. 1,315 under this construction phase. Approximately 4,000,000 cubic yards of chalk and shale were excavated from the outlet works and spillway areas and placed in the upstream and downstream embankment berms.

4.4 **Earthwork Stage III (June 1950 - March 1953).** Stage III work included construction of the left bank embankment to its full height, placement of riprap on the upstream slope of the embankment in the vicinity of the intake structure, construction of an upstream cutoff trench along the right bank of the river, removal of right bank loess material beneath the embankment and backfilling with compacted impervious fill, diversion of the Missouri River through the outlet works, construction of the embankment in the closure section and over the right bank area to a maximum elevation of 1,325, and placement of the upstream impervious blanket in the closure section and over the right bank area. In addition, the approach and discharge channels of the outlet works were completed and a portion of the spillway channel was excavated. The outlet works structures, including the tunnels, intake structures, downstream outlet structure and powerhouse substructure were under a separate contract and were sufficiently complete to allow diversion of the river. Approximately 7,350,000 cubic yards of compacted fill, 2,060,000 cubic yards of hydraulic pervious fill in the closure foundation area, and 194,000 cubic yards of hydraulic chalk fill in the diversion weir structure were placed during this construction stage. In addition, approximately 3,800,000 cubic yards of chalk and shale were excavated from the outlet works and spillway areas and were placed in the upstream and downstream chalk berms. Photos No. 5 through No. 18 were taken during Earthwork Stage III.

4.5 **Earthwork Stage IV (November 1952 - September 1953).** The compacted impervious and upstream chalk berm sections of the embankment were placed to Elev. 1,355 in the right bank and closure areas under the Stage IV construction. Also included was the 5-foot thick pervious drainage fill blanket beneath the right bank downstream compacted section of the impervious embankment. Approximately 2,740,000 cubic yards of impervious fill and

220,000 cubic yards of pervious fill were placed in the compacted earthfill portions of the embankment. About 3,590,000 cubic yards of chalk and shale were excavated from the spillway area and were placed mostly in the upstream chalk berm section of the embankment. Photos Nos. 19 and 20 were taken during Earthwork Stage IV.

4.6 **Earthwork Stage V (July 1953 - November 1955).** Earthwork Stage V was the final embankment construction stage. The embankment over the right bank and closure section was raised to its design crest elevation of 1,395. The left bank portion of the embankment had previously been constructed to Elev. 1,395 under Earthwork Stage III. The entire length of the embankment crest was overbuilt to heights up to 3.5 feet, as indicated on Plate A19, to compensate for post construction settlement. The embankment at both abutments of the spillway structure was constructed to the final crest elevation of 1,395. The left and right bank embankment toe drains were installed and 14 of the 36 previously installed relief wells were raised in conjunction with raising of the chalk berm in the downstream closure area. Excavation of the spillway discharge channel was also completed under this earthwork contract. Approximately 4,330,000 cubic yards of impervious fill and 27,000 cubic yards of pervious drain fill were placed in the compacted embankment. In addition, about 4,560,000 cubic yards of chalk and shale were excavated from the spillway area and placed in the upstream and downstream berms of the embankment. Photos No. 21 through No. 25 were taken during Earthwork Stage V.

5. **FOUNDATION PREPARATION.** All areas upon which embankment material were placed, plus at least a 10-foot contiguous strip, were cleared of all brush, trees, structures, trash, debris, and other unsuitable foundation material. Roots larger than 1-1/2 inches in diameter were removed to a minimum depth of 3 feet below the ground surface. Thin surface layers containing sod, humus, and other undesirable material were stripped and wasted. Prior to placement of embankment, the foundation was loosened to a depth of 12 inches by scarifying, plowing, or harrowing, cleared of loosened roots and debris, then compacted as for impervious fill. Foundation areas and conditions requiring

specific treatment, such as low left bank areas, left bank springs, right bank loess deposit, and the river closure area are discussed below.

5.1 **Left Bank Preparation.** Low ponded areas on the left bank were filled with pervious alluvial sands to assure a dry foundation for subsequent embankment fill operations. Placement of the sand fill was done by end-dumping and dozing the material in place, an operation that pushed water and muck away from the foundation area. When internal drainage of the fill material was impeded, the filling operation was temporarily stopped to allow dissipation of the pore pressures.

During the initial earthwork stage, the natural spring near the centerline of the dam about 400 feet from the left bank of the river was excavated of muck and provided with a drainage channel to the river. The spring area was about 145 feet in diameter. The excavation extended 5 to 8 feet in depth into sound foundation sand and was backfilled with relatively clean pervious sand and gravel.

5.2 **Removal of Right Bank Loess.** The surface loess material was removed from a major portion of the right bank embankment foundation. A loess excavation plan and sections are shown on Plate A-20 and additional profiles are on Plate A-21. The loess was removed to increase stability, reduce settlement, and minimize possible cracking of the embankment from uneven foundation settlement. The excavated material was then reused to fill the excavation as compacted impervious fill. The entire right bank excavation involved the removal of approximately 2,450,000 cubic yards of predominantly loess material.

5.3 **River Closure Area.** Immediately after diversion of the river, the embankment foundation within the river channel was prepared by filling with pervious sand. Prior to placement of the material, all chalk that was used for river bank protection within the embankment area was removed with a iragline and later placed in the upstream chalk berm. The foundation for the

upstream end of the impervious blanket was placed by hauling pervious material which was stockpiled from material excavated from the outlet works approach channel. The material was dozed into the river channel to at least Elev. 1,242, above the water level. Photo No. 11 shows placement of the sand. The pervious foundation fill in the major portion of the channel, between the upstream dumped pervious foundation and the downstream diversion dike, was placed hydraulically by dredging sand from the downstream river bed. This operation is pictured in Photo No. 14. Filling started at the usptream end and progressed downstream. The closure plan and sections are shown on Plate A-23. Approximately 1,630,000 cubic yards of pervious foundation material were placed in the river channel.

6. **MATERIALS AND MATERIALS PLACEMENT.** The embankment was constructed of material obtained primarily from required excavations from the outlet works and spillway areas. Construction operations were generally in two 10-hour shifts. Impervious glacial till material and chalk were the predominant types of material used to construct the compacted embankment and berms, respectively. Pervious sand fill was used primarily as river chute fill and as pervious drainage blanket beneath the downstream section of the embankment. Data on field compaction tests and on embankment construction were obtained from construction reports that were prepared by the Area Engineer and his staff during and immediately after construction of the various earthwork stages. Information was also extracted from laboratory test records on approximately 70 undisturbed box samples that were taken during construction of the compacted impervious embankment. Tests on the record samples included classification, moisture contents, specific gravity, and density. In addition, some of the samples were tested to determine shear strength and consolidation characteristics. Properties and placement of the embankment material are discussed below. Material types include pervious river chute fill, rolled embankment fill, impervious blanket fill, pervious drain fill, and chalk berm fill.

6.1 **Pervious River Chute Fill.** River chute filling is described in paragraph 5, Foundation Preparation. Material placed in the closure chute consisted primarily of dredged pervious alluvial sand from the riverbed located downstream of the embankment area. The hydraulically placed material was assumed to be in a similar state of compaction as the existing natural alluvial foundation. Photos No. 11 and No. 14 show placement of the river chute fill.

6.2 **Rolled Embankment Fill.** The central wide-based main embankment section was constructed of predominately impervious material and was designated as the "rolled embankment" section through Earthwork Stage III and is shown as such on Plates A-2, A-4, and A-5. In Earthwork Stages IV and V, this section was designated as "impervious fill" section. The material requirements for the section, however, were the same in all stages. Except for a relatively small quantity of excavated and recompacted right abutment terrace clay and loess, the rolled embankment section was constructed primarily of glacial till material excavated from the left abutment. Approximately 90 percent of the rolled embankment section was constructed of till material which consisted predominately of impervious sandy clays. Small quantities of relatively pervious material that were present in the excavated till were placed in the downstream one third of the rolled embankment section.

Through Earthwork Stage III, the rolled embankment material was placed, as specified, in 8-inch loose lifts and compacted by 6 passes of a sheepsfoot tamping roller. Scarifying, blading, and watering operations are pictured in Photos Nos. 2, 3, and 4 and compaction by sheepsfoot is shown by Photos Nos. 6, 12, and 13. During Earthwork Stage III, the contractor was allowed by contract modification to place the material in 12-inch lifts with compaction provided by at least 3 passes of a 50-ton pneumatic roller. Both compaction methods were allowed in Earthwork Stages IV and V. Except in very few instances, adequate compaction was obtained by the specified minimum number of compaction coverages. Both types of rollers resulted in comparable densities; however, the pneumatic roller was generally preferred by the

Contractor because of the lower number of required coverages. Moisture content was maintained near optimum and was controlled by visual inspection by experienced Corps of Engineers inspectors who had immediate access to the results of numerous ongoing field control tests, such as classification, moisture content, compaction, and density. Moisture contents were not specified in the earlier construction stages; however, in Earthwork Stages IV and V, they were specified to be not less than 2 percent below optimum and not more than that required for proper excavating, hauling, placing, and compacting without causing excessive deformation.

Numerous field tests were made during construction of the rolled embankment. For example, during Earthwork Stage II, an average of 20 field density tests were made during each 10-hour shift. In addition, an average of one sample was taken for each 12,000 cubic yards of compacted fill for determining classifications, moisture contents, densities, and air voids. Only a partial record of field control tests exists and, therefore, the total number of field control tests conducted for the project is not known. Available data on at least 1,100 tests made during Earthwork Stages III and V show that the average dry density of the rolled embankment material was about [illegible].5 pounds per cubic foot (pcf) and the average moisture content was about 18.5 percent. The material was predominantly sandy clay (CL) and compaction averaged approximately 100 percent of maximum standard density. The tests conducted during the initial Earthwork and Earthwork Stage II construction showed an average dry density of 101.9 pcf and an average moisture content of 17.7 percent.

Fifty-seven undisturbed box samples were taken of the compacted fill during the Initial, Stage II, and Stage III earthwork contracts. This averaged about one sample for each 250,000 cubic yards of compacted fill. For Stages IV and V, a total of 13 box samples were taken for an average of approximately one sample for each 550,000 cubic yards of compacted fill. All samples were shipped to the Missouri River Division Laboratory in Omaha, Nebraska for testing. The test results for the 70 record box samples are tabulated on Plate A-41. For comparison with the results of the field

control tests given above, the tests on the undisturbed samples revealed a slightly lower average dry density of 99.5 pcf and a slightly higher average moisture content of 19.9 percent. Atterberg limits are plotted on Plate A-42 and moisture - density plots are shown on Plate A-43. The direct shear test results, shown on Plate A-44, show that only one of the 44 test envelopes fell entirely below the design strength envelope. Also, the tan Ø values of all the tests ranged from 0.36 to 0.73, all higher than the 0.35 design value. These results indicate that the adopted design shear strength parameters, tan Ø = 0.35 and cohesion, C = 0.35 tons per square foot were conservatively selected.

6.3 **Impervious Blanket Fill.** The impervious blanket adjoins the upstream toe of the rolled embankment section. It consists of material similar to that used in the rolled embankment section, except that pervious material was not allowed. Impervious material that was unsuitable for the rolled section, due to excessive moisture or presence of pieces of weathered chalk, however, was allowed in the impervious blanket. The material was placed in 12-inch thick layers and was compacted by 3 complete coverages of a crawler type tractor weighing not less than 10 tons. Field compaction test records on this material are not available; however, the construction reports indicated that the impervious blanket was well compacted.

6.4 **Pervious Drain Fill.** Pervious fill was used in the downstream section of the embankment on both abutments.

6.4.1 **Left Abutment.** A 10-foot thick pervious fill blanket drains a natural basal sand stratum which overlies a small portion of the chalk bedrock in the left abutment. The pervious fill, consisting of pit-run sand and gravel free from overburden soils was placed during the initial earthwork stage at the location shown on Plate A-15. The blanket lies between Elev. 1,320 and Elev. 1,330 and extends from Sta. 105+90 to Sta. 107+90. It outlets at the downstream toe of the embankment into a collector drain which was installed under Earthwork Stage V. The material was placed in 12-inch thick layers and compacted with a crawler-type tractor weighing at least 10 tons.

6.4.2 **Right Abutment.** A 5-foot thick pervious blanket was constructed beneath the downstream embankment section. It was constructed during Earthwork Stage IV and extends from Range 5,050 (50 feet downstream of the dam axis) to the downstream toe of the embankment and from Sta. 29+00 to Sta. 57+15. The material consisted of clean, free-draining sand, and sand and gravel containing not more than 10 percent of particles finer than a standard No. 200 sieve. It was obtained from onsite stockpiled excavated sand. The specifications required a lift thickness not exceeding 12 inches for compaction by 6 passes of a tamping roller and not exceeding 18 inches for compaction by 3 passes of a rubber-tired roller. If adequate compaction was not obtained by these methods, the contractor was required to make 3 complete coverages with a 10-ton crawler type tractor. Daily summary records of field compaction tests indicated that final compaction was made with a crawler tractor. The dry density of the surface layer averaged about 109.5 pcf and that of the underlying layer, at least a foot below the surface, averaged about 112.5 pcf. This reflects the additional compaction received by the underlying layer through surface rolling. Moisture contents generally were between 6 and 10 percent. A toe drain collector pipe was later installed along the downstream end of the pervious blanket during earthwork Stage V.

6.5 **Chalk Berm Fill.** Chalk and shale excavated from the outlet works and spillway were used to form the massive upstream and downstream chalk berms. Berm construction continued through all of the earthwork construction stages. The excavated chalk was generally blocky and varied in size from about 5 feet to a fine granular particle. The total chalk-shale excavation included relatively small quantities of shale. This was due to shale being encountered immediately below the overburden in the upper left abutment, mostly upstream of the spillway and in the approach channel. Most of the shale that was excavated in the initial earthwork stages was placed in the downstream chalk spoil area that formed part of the right bank area of the outlet works discharge channel. In later earthwork stages, the shale was placed primarily in the upstream chalk berm. The chalk berms, therefore,

were constructed predominately of chalk material, which is more resistant to weathering than shale. The berms were constructed generally in lifts not exceeding 10 foot in thickness, with no moisture control, and compaction only by the hauling and grading equipment. Later exposure of the upper 10 foot of the upstream chalk berm by wave erosion revealed that the material was moderately compact with no visible voids. The upstream chalk berm, with an outer slope of 1V on 15H, was considered during project design to be an adequate wave protection alternative to the more expensive conventional riprap protection. Wave erosion scarps, however, developed and required stone protection, as described in paragraph 7, Wave Protection.

Two test embankments, one of shale and the other of chalk, were constructed under Modification No. 4 of the initial earthwork contract. The shale test embankment was constructed in the upstream blanket area between Sta. 86+25 and Sta. 90+25 and Range 3,925 and Range 4,175. The chalk test embankment was made in the chalk spoil area on the right bank of the outlet works discharge channel. The test embankments were constructed to determine the compaction characteristics of shale and chalk which were then being considered for possible use in the rolled embankment section. This possibility was prompted by an expected decrease in overburden excavation due to a revision in the spillway excavation plan. The test fills showed that compaction of these materials into an impervious mass required a rather long and expensive process. The process involved breaking the large chunks by several passes of a specially built spike-toothed roller, then compacting with a sheepsfoot roller. Although density and unconfined compression tests were performed on samples from the compacted fill, additional tests were desired but not performed to determine the effects of saturation and long term weathering of the materials. For these reasons, the tests were considered inconclusive and the use of shale and chalk was not considered further for use in the rolled embankment section. As it later turned out, ample quantity of overburden was available for the construction of the rolled impervious section. The results of the test embankments were later supplemented by additional tests and were used in the design and construction of Gavins Point Dam.

7. WAVE PROTECTION. Riprap protection on the upstream slope of the embankment was originally placed only in the vicinity of the outlet works intake structure and spillway. Later placement of stones on the embankment upstream chalk berm was made only after many years of observation and monitoring strongly indicated that the wave erosion scarps would continue to increase in height and eventually cut into and adversely affect the stability of the main embankment.

7.1 Riprap at Intake Structure and Spillway. The entire upstream slope of the embankment in the vicinity of the outlet works intake structure is protected with riprap. The riprap extends from Sta. 103+00 to the right spillway wall, at approximately Sta. 121+40. The riprapped slope from Sta. 103+00 to Sta. 106+00 extends beneath the northeast end of the upstream chalk berm. This was done for protection of the rolled embankment as some erosion of the chalk berm was expected to occur in this area. Riprap also protects the left bank of the spillway approach channel. The riprap near the intake structure was placed during Earthwork Stage III and those adjacent to both spillway walls were placed later during Earthwork Stage V. The riprap section near the intake structure and right spillway wall consists of a 3-foot layer of riprap, a 1-foot layer of spalls, and a 2-foot layer of filter blanket as indicated on Plates A5 and A-16. The riprap at the left spillway wall is similar, except that a 1-foot filter blanket is used. The riprapped slopes are in good condition.

7.1.1 Riprap. Riprap consists of glacial boulders that were stockpiled from required left abutment excavations. The stones were reasonably well-graded from a 3-foot maximum size to a 5-inch minimum size.

7.1.2 Spalls. The spall layer was constructed by a subcontractor who obtained the material from a locally owned gravel pit. The stones were reasonably well-graded between the 6- and 2-inch sizes.

7.1.3 **Filter Blanket.** The filter blanket underlying the spalls was also constructed by the spalls subcontractor. The processed material was obtained from a locally owned gravel pit and was required to meet the following gradation.

Sieve Size U.S. Std. Sq. Mesh.	Percent by Weight Passing
3-1/2-inch	100
3-inch	95-100
2-inch	90-100
3/4-inch	75-100
3/8-inch	60-100
No. 4	40-80
No. 10	25-60
No. 16	20-50
No. 100	0-15

7.2 **Stone Protection on Chalk Berm.** The flat 1V on 15H upstream chalk berm was assumed to provide adequate resistance against wave action. However, the berm started to erode when the reservoir was initially raised to Elev. 1,355 in 1957. Erosion continued into the 1V on 7H chalk slope above the flatter berm and progressed toward the main embankment section at an average rate of about 6 feet per year. By 1973, it had advanced to about range 4,850, approximately 150 feet from the axis of the dam. Scarps up to about 10 feet in height formed along practically the entire length of the unprotected embankment slope.

In 1977, the 8,000-foot long erosion scarp was graded to a 1V on 3H slope, but stone protection was placed only along a 400-foot reach adjacent to the existing riprap in the vicinity of the intake structure. The stone protection consisted of dumped field boulders having a maximum size of 250 pounds and a median size of 50 to 100 pounds and placed at a rate of 2.5 tons per lineal foot of scarp. The unprotected graded slope was severely eroded by waves in 1978 which led to the placement of riprap over the entire length of the erosion scarp in 1981. Riprap composed of field boulders was dumped at an average rate of 2.5 tons per lineal foot of scarp length and consisted of stones having a maximum size of 400 pounds and a median size of 70 to 150 pounds. These stones are about 50 percent larger than those used in 1977, and were selected for use after excessive stone displacements were noted in the existing riprap.

8. **DIVERSION AND CLOSURE.** Prior to construction of the embankment closure section, the Missouri River was fully diverted through the outlet works on 20 July 1952 as part of the Earthwork Stage III construction. Photo No. 15 shows an aerial view of the diverted river. Diversion was effected by dredging chalk directly into the river. Location and typical sections of the diversion structure are shown on Plate A-22. As shown in Photo No. 8, the chalk was dredged from the chalk spoil area located downstream of the dam and riverward of the outlet works discharge channel. The diversion plan was developed by the contractor as an alternative to the plan shown on the contract drawings which required the use of lumber mattresses, timber cribs, and an open deck pile trestle.

The diversion dam consisted of a lower, wide-based chalk blanket, a dredged chalk weir section, and an upper dumped chalk dike. Stage construction of the dam is indicated by the sections shown on Plate A-22. The chalk blanket was placed to Elev. 1230 approximately 10 feet below the river level, from 29 April 1952 through 22 May 1952. Construction of the dam was temporarily suspended until 7 July 1952 while the outlet works approach and discharge channel plugs were being removed. The diversion weir section was then constructed from 8 July 1952 through 20 July 1952 to Elev. 1244, about 2 feet above the headwater level, to complete the river diversion. By 29 July 1952, the diversion structure was completed by end dumping chalk directly onto the weir section to Elev. 1265. Photo No. 9 shows placement of dredged chalk for the construction of the diversion weir and Photo No. 10 shows end dumping operations in building the dike over the weir section.

Immediately following the diversion operations and prior to placement of fill in the embankment closure section, a dike was first constructed along the upstream end of the impervious blanket foundation. The dike was constructed by end dumping sand that was previously stockpiled from the outlet works approach channel excavation. This operation is shown in Photo No. 11. The foundation for the upstream impervious blanket and the main embankment was then hydraulically filled with sand dredged from the downstream riverbed. Hydraulic fill operations are shown in Photo No. 14.

Following placement of the sand foundation to above water level, the impervious blanket was constructed to Elev. 1,265 and the rolled embankment was constructed to Elev. 1,325. The closure plan and section are shown on Plate A-23. The rolled embankment section over the right abutment and closure area was subsequently raised to Elev. 1,355 under Earthwork Stage IV and brought up to final crest, Elev. 1,395, under Earthwork Stage V, the final embankment construction stage.

9. **SEEPAGE CONTROL.** The embankment is founded on a deep deposit of alluvial sand in the valley and primarily on natural or recompacted clay overburden in the abutments. Seepage through and beneath the valley embankment section is controlled primarily by the massive embankment and berm sections and by pressure relief wells along the downstream toe of the compacted embankment. Cutoff walls and trenches were considered for underseepage control across the valley section, but were rejected primarily because of the high cost of these methods. Seepage control on the right abutment included removal and recompaction of surface loess deposits, construction of an upstream cutoff trench, and construction of a downstream pervious drain blanket. Control on the left abutment included treatment of a pervious glacial stratum and blanketing of pervious exposures in the approach channel.

9.1 **Valley.** Seepage control in the valley is provided by the massive embankment section and by pressure relief wells.

9.1.1 **Embankment Section.** The massive embankment section, including the central compacted impervious section, the upstream impervious blanket, and the upstream and downstream chalk berms, provides the necessary seepage resistance to keep hydrostatic uplift pressures to below the levels assumed during design. The downstream chalk berm provides stability of the area downstream of the compacted embankment section.

9.1.2 **Relief Wells.** Thirty-six relief wells were installed along the downstream toe of the impervious embankment section to control uplift pressures that develop in the alluvial foundation, especially the

higher pressures that develop at lower depths. The wells were designed to discharge flows directly through the screens and into the chalk fill berm. The berm was assumed to contain sufficient voids to accommodate the discharges. The location of the wells are shown on Plate A-11. The wells are spaced at 60 to 115 feet, but are mostly at 100-foot intervals. They are fully penetrating, except in the center of the valley where bedrock is at a considerable depth. In this deeper section, the well screens were set at about Elev. 1,150 resulting in a well penetration of approximately 60 percent. Relief wells RW-1 through RW-14 were extended by the Earthwork Stage V contractor in conjunction with raising of the downstream chalk berm in the embankment closure section. The riser and screen sections of the relief wells consist of 8-inch inside diameter wire-wrapped wood stave pipes. Each well includes a 36-inch diameter corrugated metal pipe well pit. A tabulation of the relief wells and relief well piezometers and a soils profile along the line of wells are shown on Plates A-12 and A-13. Relief well and piezometer details are on Plate A-14. Relief well design computations are presented on Plates A-38 and A-39 and the gradation curves of the gravel packs are on Plate A-40.

Twenty-one well point type piezometers were installed between and in line with the relief wells to monitor the uplift pressures at the line of wells. These piezometers are described in "Instrumentation," below. The water level in the wells and the piezometers have always remained below the top of the screen which is at about Elev. 1,270. Distortion of the riser and screen by settlement of the chalk fill prevented flow measurements from being made in 24 of the 36 wells, including all of the wells in the closure area. In wells where the flow meter could be used, only insignificant or no flows were indicated. Inspection of the interior of the pipes with the aid of a mirror revealed the bulging of the pipes and indicated the water surface to be generally still and without ripples. Water levels in the wells and piezometers fluctuate with changes in pool levels, but the average readings have varied little over the years. Detailed description and evaluation of the relief well system are presented in the referenced "Embankment Relief Well Study" report.

9.2 **Right Abutment.** Seepage control measures at the right bank included excavation of the natural loess and backfilling with impervious material, construction of a cutoff trench along the right bank, and construction of a pervious drain blanket beneath the downstream section of the embankment.

9.2.1 **Loess Excavation.** Loess excavation is described in paragraph 5, Foundation Preparation. In addition to preventing excessive settlement of the overlying embankment, removal of the loess and its replacement with compacted impervious material resulted in a stable impervious foundation.

9.2.2 **Cutoff Trench.** A cutoff trench was excavated through the sand and gravel layer that overlies the Niobrara Chalk along the right bank. The location and details of the trench are shown on Plates A-20 and A-21. Dewatering was required in the deeper excavations and the trench was backfilled with compacted impervious material. The loess covered area between the cutoff trench and the main embankment was excavated to a depth of 5 feet and was then backfilled with the recompacted excavated material to form a continuous relatively impervious blanket.

9.2.3 **Pervious Drain Blanket.** The 5-foot thick pervious drain blanket beneath the downstream embankment section on the left abutment is described in paragraph 6.4.2. Its purpose is to increase the stability of the embankment by providing drainage for both the overlying embankment and underlying foundation. The blanket is provided with a toe drain collector pipe along the downstream toe of the embankment. Details of the toe drain are shown on Plates A-18 and A-19. The toe drain pipe outlets into the massive chalk berm through a perforated end section. The toe drain has shown very little or no seepage flows.

9.3 **Left Abutment.** Seepage control at the left bank consisted of seepage cutoffs and drainage of the pervious glacial stratum and also impervious blanketing of the pervious sand exposures along the outlet works approach channel.

9.3.1 Pervious Glacial Stratum. Treatment of the pervious stratum on the left abutment is detailed on Plate A-15. The glacial drift and basal sand overburden beneath the upstream section of the embankment was removed to the top of the chalk formation and was replaced with rolled impervious embankment material. A shallow cutoff trench was excavated into the top of the chalk formation. Drainage of the basal sand is through the pervious blanket and is described in paragraph 6.4.1. A 12-inch diameter perforated CMP at the downstream toe of the embankment collects and conducts drain water from the pervious blanket. The perforated toe drain pipe extends a distance of about 1,460 feet, from manhole No. 8 at the right spillway wall to manhole No. 5. From manhole No. 5, the drain outlets through a series of solid CMP, drop inlets, and manholes into the surface drainage system at Sta. 104+95 and Range 5,304. A plan and details of the toe drain are presented on Plate A-17.

9.3.2 Blanketing of Approach Channel. Sand and gravel exposures along the outlet works approach channel were excavated, then covered with a 15-foot thick compacted impervious blanket. The locations of these exposures are indicated on Plate A-3 and a typical blanket section is depicted by section H-H shown on Plate A-5. Seepage analyses during design revealed that exposure of the sand layer in the approach channel would create the shortest underseepage path beneath the embankment. Water entering the sand would seep around the riverside of the outlet works tunnels and into the discharge channel. Relief wells and drains located riverward of the outlet works and discharge channel provide additional underseepage control in this area.

10. EMBANKMENT STABILITY. The stability of the embankment over the right bank terrace clay, the valley alluvium, and the left bank glacial material were analyzed during the project design stage.

10.1 Stability of Embankment Over Right Bank Terrace Clay. A "wedge" method of stability analysis was performed for the embankment founded over

the thick clay deposit on the right bank terrace. The analysis is graphically shown on Plate A-35. With impervious layers above and below the saturated clay, drainage would be greatly impeded and the pore pressure due to the fill load would be dissipated very slowly. For the stability analysis, very little consolidation of the clay foundation was assumed to occur during construction. The minimum factor of safety obtained was 2.2 which was believed to be adequate for the method of analysis used. A reanalysis in 1952 during construction of the project indicated a reduced factor of safety of 1.4, primarily due to the use of a higher ground water level and also to minor changes in the embankment section. The ground water level was raised from Elev. 1,245 to Elev. 1,275. The toe of the embankment was changed to a 1V on 3H slope instead of a feathered slope, and the pervious drain blanket did not extend as far upstream as in the original embankment section analyzed.

10.2 **Stability of Embankment Over Valley Alluvium.** The embankment section overlying the valley alluvium was analyzed during the initial design stage by the "wedge" method, as graphically presented on Plate A-36. The section did not include the chalk berms as the extent of the berms were not known at that time. When the analysis was made, test results were not available on the strength of the shale foundation. The required shear strengths computed for safety factors of 1.5 and 2.0, assuming zero cohesion, were tan ϕ = 0.189 and tan ϕ = 0.358, respectively. Consolidated direct shear tests later indicated a tan ϕ = 0.3 and cohesion = 0.1 tons per square foot for the shale. On this basis, a factor of safety of about 1.8 was assumed for the analysis. The safety factor is actually much higher due to the presence of the chalk berms.

The same section was also analyzed using a modification of the Fort Peck Elastic Theory method, the results of which are shown on Plate A-37. In this method, the horizontal shearing forces along the shale-sand contact were computed for a depth of 75 feet below the structure. These were determined assuming that no stress discontinuity occurs at the contact plane or that the

shale might act as a rigid boundary. It is believed that actual conditions are somewhere intermediate between these assumptions. Using the rigid boundary assumption, the minimum safety factor against horizontal sliding along the contact at any point was found to be 1.7. On the other hand, the infinite depth assumption gave a minimum point safety factor of 2.5.

10.3 **Stability of Embankment Over Left Bank Glacial Till.** Details on the stability analysis for the embankment over the left abutment glacial material are not available. However, from the minutes of the 9 October 1947 Board of Consultants meeting, no great concern was expressed on the embankment stability over the left abutment. It was noted that the relatively compact material was stable and that based on extremely conservative assumptions, stability studies indicated a safety factor of 1.5. It was also stated that a safety factor of 1.3 was obtained for the sudden drawdown condition.

11. **SETTLEMENT.** To compensate for expected settlement, the crest of the embankment was overbuilt during Earthwork Stage V, the last earthwork contract stage. Overbuild profiles and details are shown on Plate A-19. The profile reflects the settlement estimated for the different foundation conditions beneath the right bank terrace, closure, and left bank sections of the embankment. The settlement of the right bank terrace clay was estimated primarily on data from consolidation tests run on the clay samples. It was estimated that about 20 percent of the total settlement would occur during construction. For the closure and left bank sections, the settlement of the alluvial foundation was based on data from settlement gages that were installed during the earlier earthwork construction stages. The data indicated that 60 to 75 percent of the settlement would occur during construction.

Plots of settlement gage readings indicate that all foundation settlement have essentially stabilized and that the post construction settlement very closely followed those estimated. Consolidation of the right bank

terrace clay, however, occurred at a faster rate than predicted. By the end of construction, approximately 60 percent of the settlement of the clay foundation had occurred. The alluvial foundation settled at a rate slightly higher than predicted. Approximately 75 to 85 percent of the settlement beneath the closure and left bank section occurred during construction.

12. **INSTRUMENTATION.** Instrumentation of the Fort Randall embankment consists of embankment piezometers, settlement gage piezometers, relief well piezometers, settlement gages, crest and slope movement markers, tiltmeters (slope indicators) and strong motion accelerographs.

12.1 **Piezometers.** A general tabulation of all the piezometers for the project, except for the settlement gage piezometers, is shown on Plates A-9 and A-10. Sixty well-point type piezometers and 34 open pipe combination piezometer - settlement gages are used to monitor the hydrostatic uplift pressures beneath the embankment. The piezometers that are located beneath the main rolled impervious embankment section are described below under "embankment" piezometers. The piezometers that are located between and in line with the relief wells are discussed under "relief well" piezometers. The few piezometers that are located in the downstream area of the chalk berm are discussed under "downstream" piezometers. Details of a well-point piezometer are shown on Plate A-14.

12.1.1 **Embankment Piezometers.** There are 32 well-point type piezometers and 34 open-pipe settlement gage piezometers that extend beneath the rolled embankment section. Twenty-four of the well-point piezometers are located in seven piezometer lines, A, B, C, D, E, F, and L across the embankment, as shown on Plate A-11. The piezometers measure seepage pressures in the downstream pervious drain in both abutments and underseepage pressures in the valley alluvial sands. The abutment piezometers also give an indication of the effectiveness of the upstream cutoff trenches in both abutments. Typical piezometer plots are presented on Plates A-47 through A-52.

Eight piezometers, FR 79-1 and FR 79-3 through FR 79-9, are located beneath the downstream slope of the valley embankment section and are used to measure the uplift pressures in the alluvial sand foundation. These piezometers were placed in holes that were drilled to obtain information for a seismic evaluation of the embankment foundation.

The 34 settlement gage open-pipe piezometers are located at the settlement gage locations shown on Plate A-53. All of the settlement gage plates are set on top of the embankment foundation. The settlement gage pipe is used as an open-pipe piezometer by allowing entrance of water through small perforations in the lower 2-foot length of the 2.5-inch diameter pipe. Pervious sand was placed around the perforated pipe to at least 3 feet laterally and to at least 1 foot above the top of the perforations.

12.1.2 **Relief Well Piezometers.** Twenty-one well-point type piezometers are located between and in line with the relief wells to monitor the effectiveness of the relief well system along the downstream toe of the rolled embankment. The piezometer locations are shown on Plate A-11 and profiles of the piezometers and wells are presented on Plates A-12 and A-13. Typical plots of relief well and piezometer readings are shown on Plates A-45 and A-46.

12.1.3 **Downstream Piezometers.** Seven piezometers were installed through the downstream portion of the chalk berm to monitor uplift pressures in the alluvial sand near the toe of the chalk berm.

12.2 **Settlement Gages.** Thirty-nine settlement gages were installed during construction of the embankment. Thirty-four are still operational. The five gages at Range 3,735 were inundated by the reservoir and have been abandoned. Location and elevation data on the active gages are tabulated on Plate A-53. Each settlement gage consists of a 6-foot diameter, 1/2-inch thick steel base plate and a vertical 2-1/2-inch diameter steel pipe which

extends from the base plate about 3 feet above the embankment surface. The pipe was extended inside of a 4-inch diameter protective, steel pipe from about 6 feet above the base plate and both pipes are capped above the embankment surface. The base plate is founded on 12 inches of a sand levelling layer about 3 feet below the top of the foundation. The lower 2-foot length of the 2-1/2-inch diameter pipe is perforated and backfilled with sand to allow the pipe to act as an open-pipe piezometer. Settlement gage readings have been taken at regular intervals and typical plots are shown on Plate A-54 for the gages at Sta. 22+50, 30+00, and 40+00 or the right abutment and on Plate A-55 for gages at Sta. 82+00 and 90+00 in the valley section.

12.3 **Crest and Slope Movement Markers.** The locations of 28 crest and slope movement markers are shown on Plate A-56. Initially, the markers consisted of concrete monuments extending approximately 5 feet into the embankment. Concrete monuments were also set in the abutments for survey reference points. All of the markers were replaced with deeper markers in 1979. The new markers are of two types. One type consists of an 11-foot long, 2-inch diameter pipe set in a 10-inch diameter, 10-foot deep augered hole. It is centered in an 8-inch diameter casing which is set about 3 feet above the bottom of the pipe. The uncased pipe and the lower 1.5 foot of casing is embedded in concrete and the top of the casing is capped with a removable cover. The second type of marker consists of a 1-inch diameter, 10-foot long, steel rod driven to a depth of 9 feet below ground surface through a 4-inch diameter, 3-foot deep cased auger hole. The top of the casing extends about a foot above the ground and is provided with a removable cap. Typical plots of movement marker readings are shown on Plates A-57 and A-58.

12.4 **Tiltmeters.** Two tiltmeter (slope indicator) wells were installed in 1978 and 1979 through the embankment and underlying clay foundation in the right abutment. Tiltmeter Well T 40/503, located at Sta. 40+00 and Range 5,030, is 303.4 feet deep and extends approximately 93 feet into bedrock.

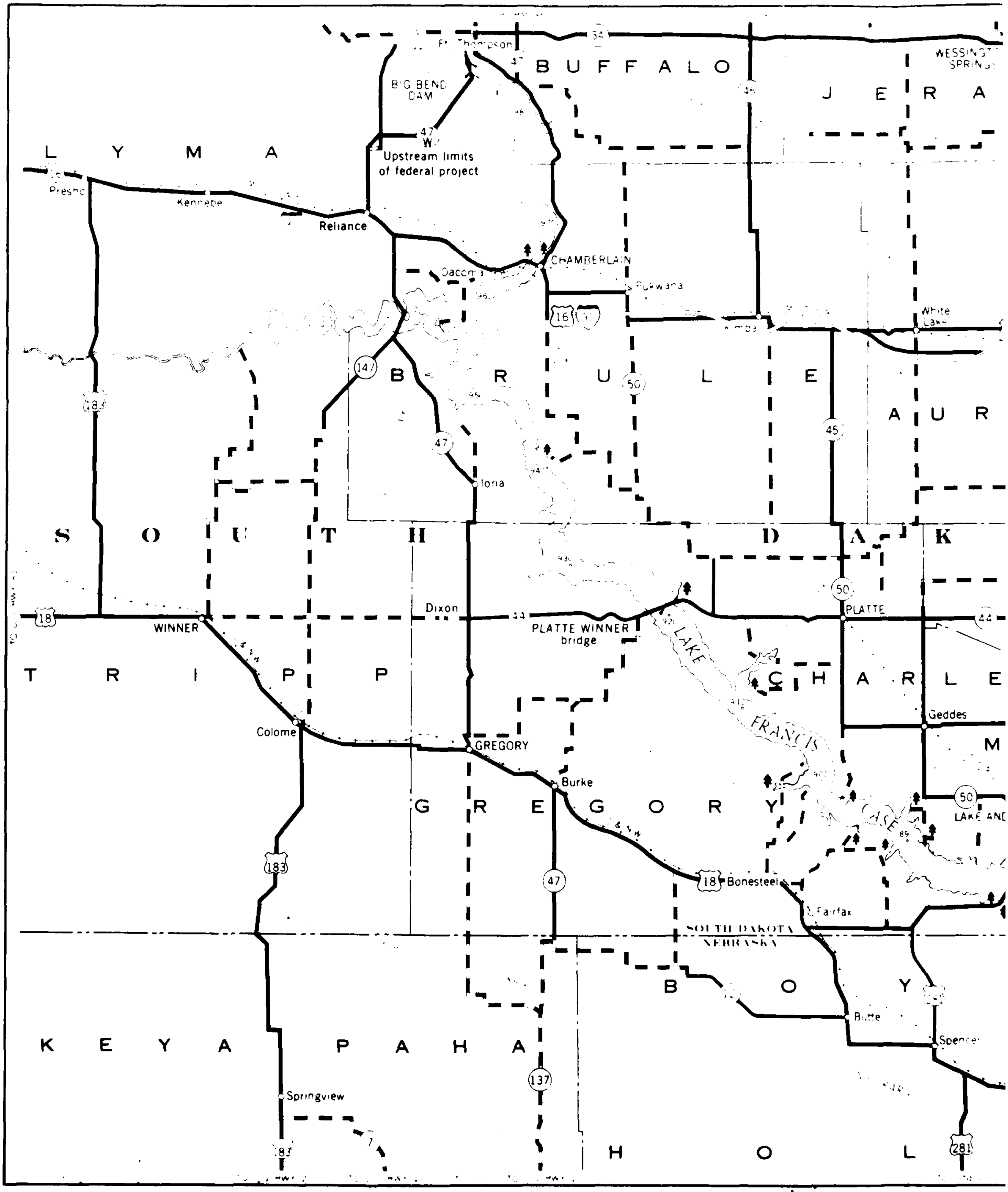
Tiltmeter Well T 40/517, located at Sta. 40+00 and Range 5,170, is 221.5 feet deep and is embedded approximately 61 feet into bedrock. Each tiltmeter well consists of a 3-inch I.D. grooved tiltmeter casing set with an 18-inch stickup in a sand or grout filled vertically drilled hole. These wells are for the purpose of monitoring subsurface displacement by allowing measurement of the change in well casing tilt with a tiltmeter. The tiltmeter casing and tiltmeter (Digitilt TM) were purchased from the Slope Indicator Company. Typical computer plots of tiltmeter readings for T 40/503 are shown on Plate A-59.

12.5 **Strong Motion Accelerographs.** Fort Randall Dam is in Zone 1, a low seismic activity region outlined in the seismic probability map, Figure 6, EM 1110-2-1902. Three Kinemetrics SMA-1 strong motion accelerographs were installed at the project in 1976. One instrument is located off the crest of the dam at about Sta. 60+00, at about the maximum section of the embankment. Another is in the downstream area near the old Fort Randall chapel, and the third is located in the west end of the spillway gallery. All of the instruments were installed and are maintained by the U.S. Geological Survey.

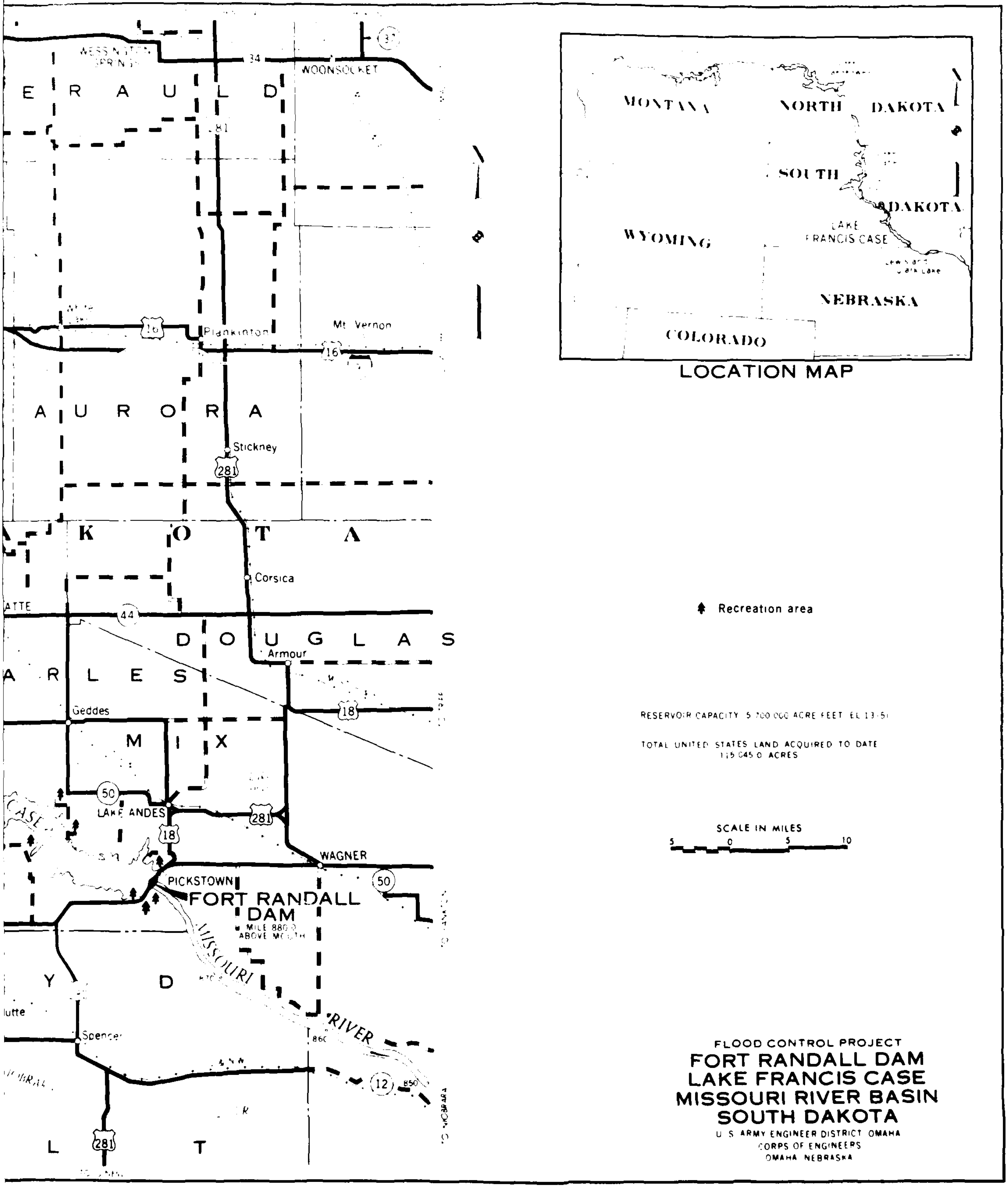
13. **OPERATIONS AND INSPECTIONS.** The Fort Randall Dam - Lake Francis Case project is operated and maintained by the U.S. Army Corps of Engineers, Omaha District. The project office is located in the powerhouse complex and is staffed by permanently assigned operations and maintenance personnel. Annual inspections of the project are conducted by personnel of the district office and periodic in-depth inspections are made jointly by members of the Omaha District and the Missouri River Division, and occasionally the Office of the Chief of Engineers. These inspections are made to assure the structural and operational soundness of this multipurpose dam project. Periodic inspections are made in accordance with the requirements of ER 1110-2-100 and to date, such inspections have been successfully conducted in 1967, 1971, 1976, and 1981. Results of the inspections are included in the referenced periodic inspection reports.

14. **EVALUATION.** The Fort Randall Dam embankment is in good structural condition. In over 28 years of operation, no serious stability problems have occurred. Instrumentation readings indicate that settlement of the embankment foundation has essentially stabilized, that no unusual embankment deformations are occurring and that hydrostatic uplift pressures are lower than those assumed during design of the project. Daily surveillance by project personnel and annual and periodic inspections by members of the District and Division offices assure that the performance of the dam is adequately monitored and evaluated.

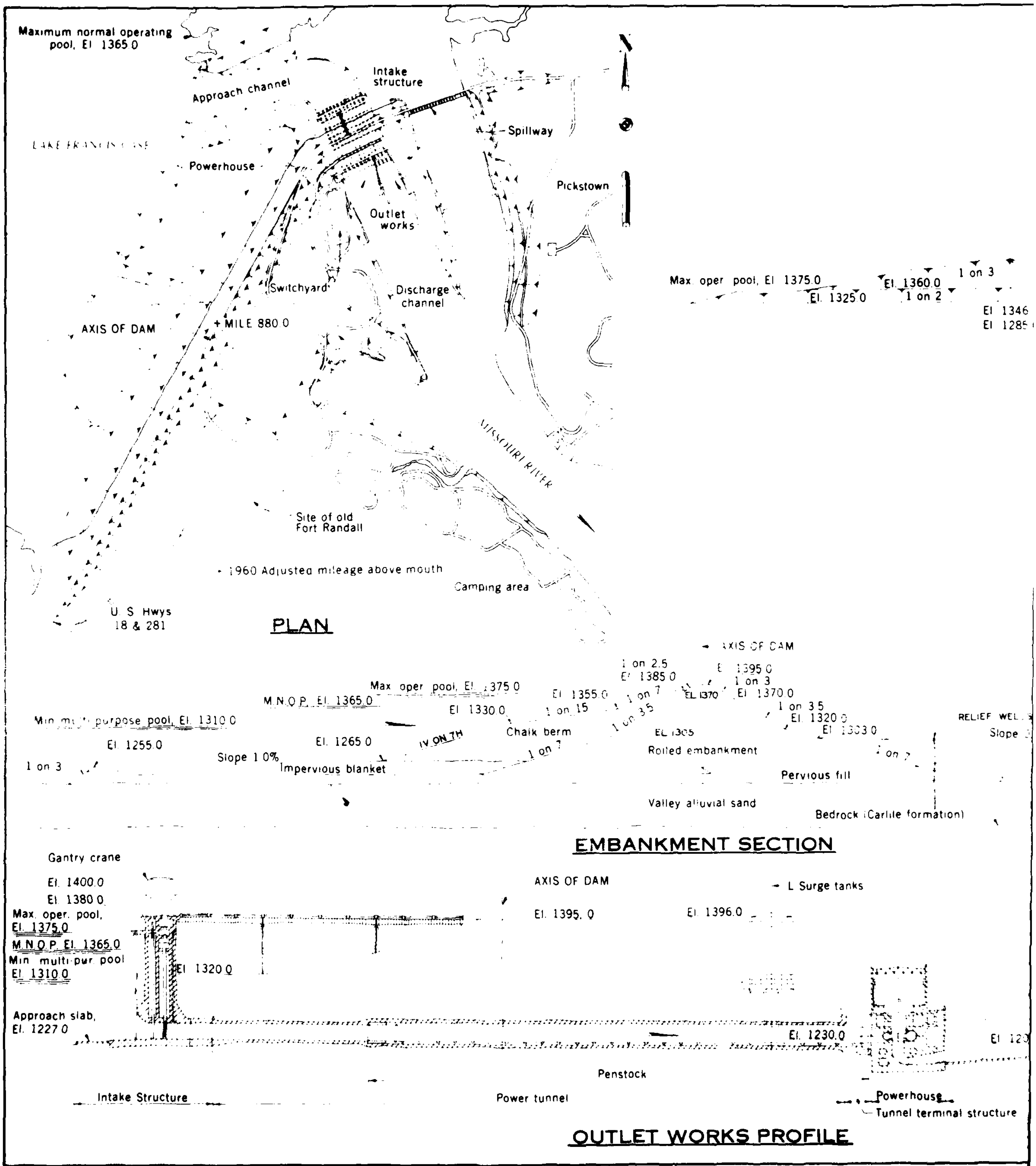
APPENDIX A
DRAWINGS


BUFFALO
JERA
LYMA
BIG BEND DAM
Upstream limits of federal project
Reliance
Kennebec
CHAMBERLAIN
Pukwana
BRULE
AUR
White Lake
Iona
SOUTH DAKOTA
Dixon
WINNER
PLATTE WINNER bridge
PLATTE
LAKE FRANCIS CASE
TRIPP
CHARLE
Geddes
Colome
GREGORY
Burke
GREGORY
LAKE AND
Bonesteel
Fairfax
SOUTH DAKOTA
NEBRASKA
BOY
Butte
Spencer
KEYA PAHA
Springview
HOL

1


LOCATION MAP
MONTANA
NORTH DAKOTA
SOUTH DAKOTA
WYOMING
NEBRASKA
COLORADO
LAKE FRANCIS CASE
Recreation area
RESERVOIR CAPACITY 5,700,000 ACRE FEET EL 1375
TOTAL UNITED STATES LAND ACQUIRED TO DATE 115,045.0 ACRES
SCALE IN MILES
5 0 5 10
WOONSOCKET
Plankinton
Mt. Vernon
AURORA
Stickney
Corsica
DOUGLAS
Armour
Geddes
LAKE ANDES
WAGNER
PICKSTOWN
FORT RANDALL DAM
MILE 880.0 ABOVE MOUTH
MISSOURI RIVER
Spencer
FLOOD CONTROL PROJECT
FORT RANDALL DAM
LAKE FRANCIS CASE
MISSOURI RIVER BASIN
SOUTH DAKOTA
U.S. ARMY ENGINEER DISTRICT OMAHA
CORPS OF ENGINEERS
OMAHA NEBRASKA

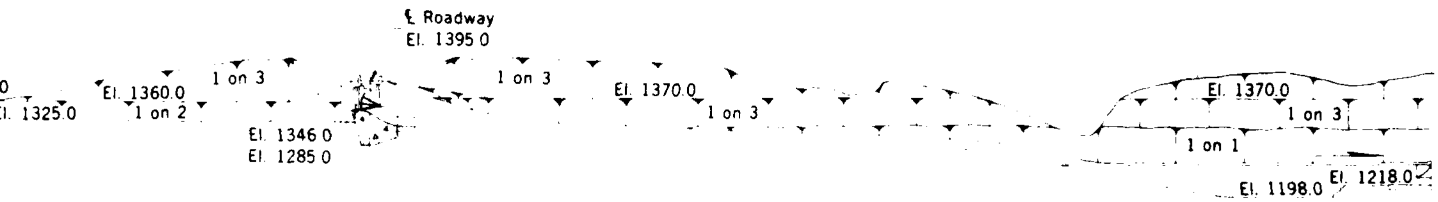
2



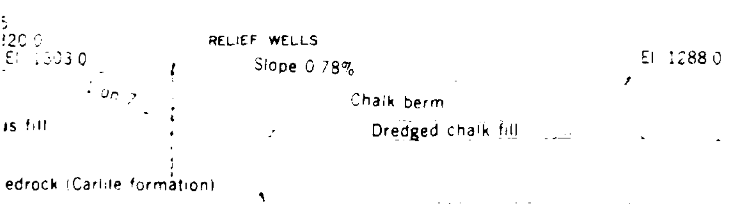
PLAN

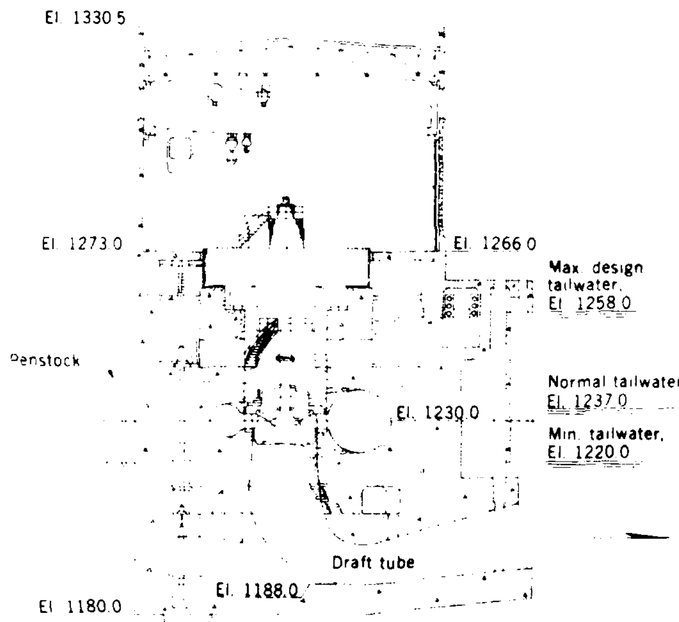
EMBANKMENT SECTION

OUTLET WORKS PROFILE

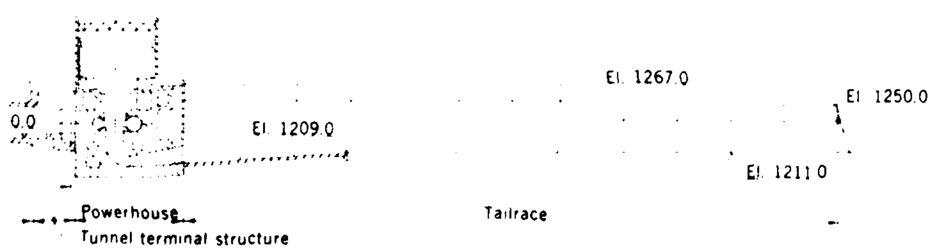


SPILLWAY PROFILE

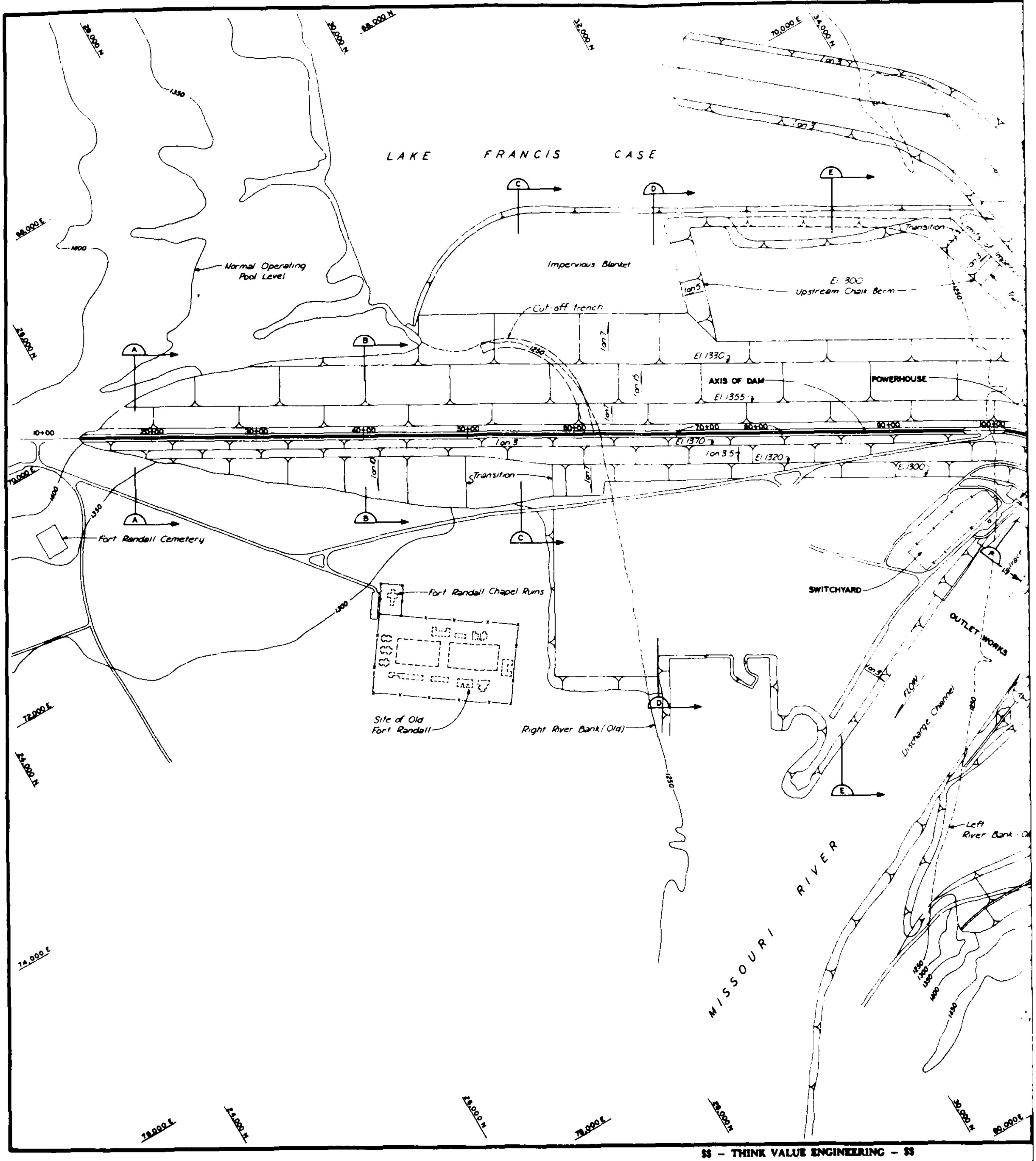




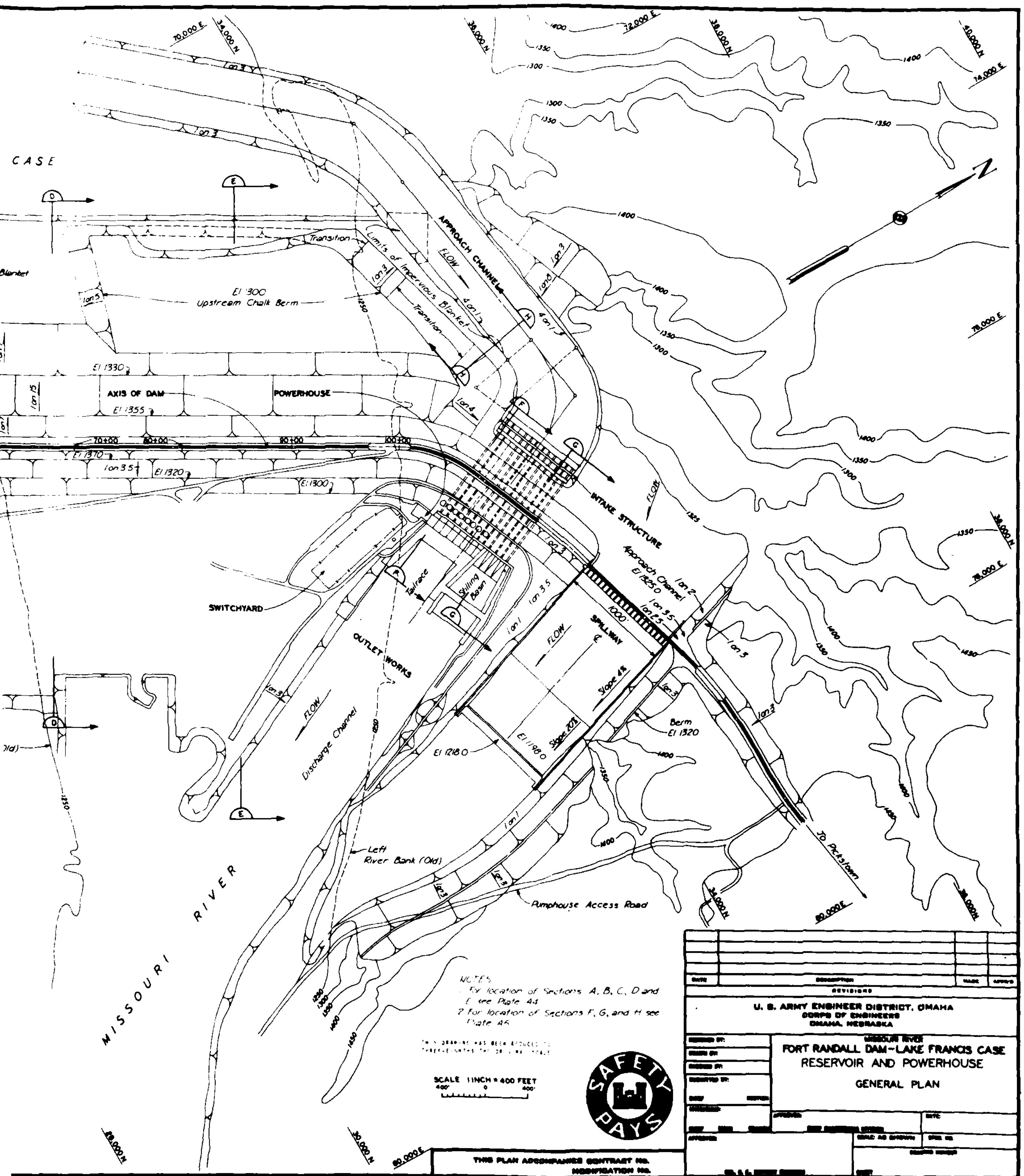
POWERHOUSE SECTION



FLOOD CONTROL PROJECT
FORT RANDALL DAM
LAKE FRANCIS CASE
MISSOURI RIVER BASIN
SOUTH DAKOTA
U.S. ARMY ENGINEER DISTRICT OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA



$$ – THINK VALUE ENGINEERING – $$

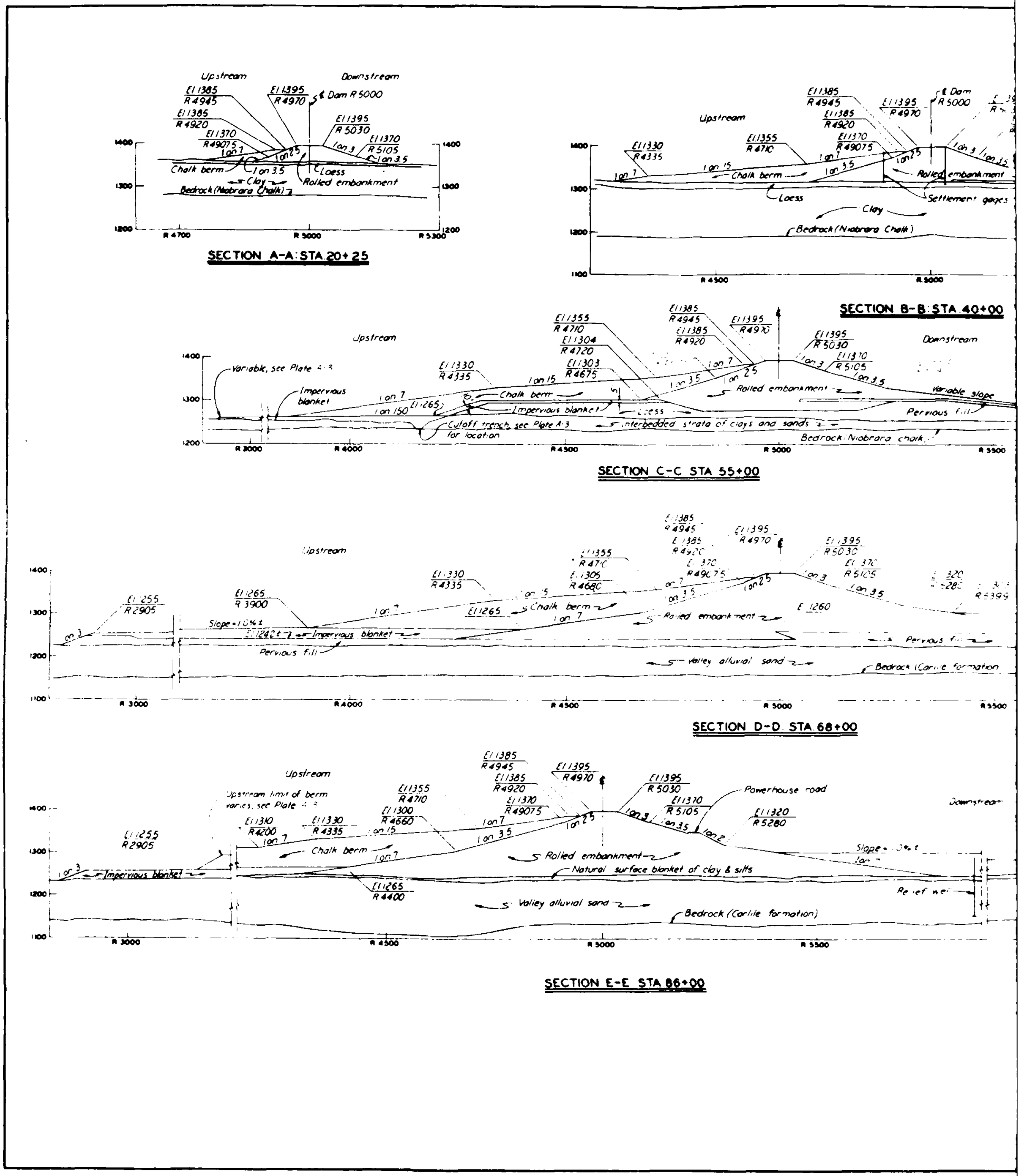


$$ – THINK VALUE ENGINEERING – $$

EMBANKMENT CRITERIA AND PERFORMANCE REPORT

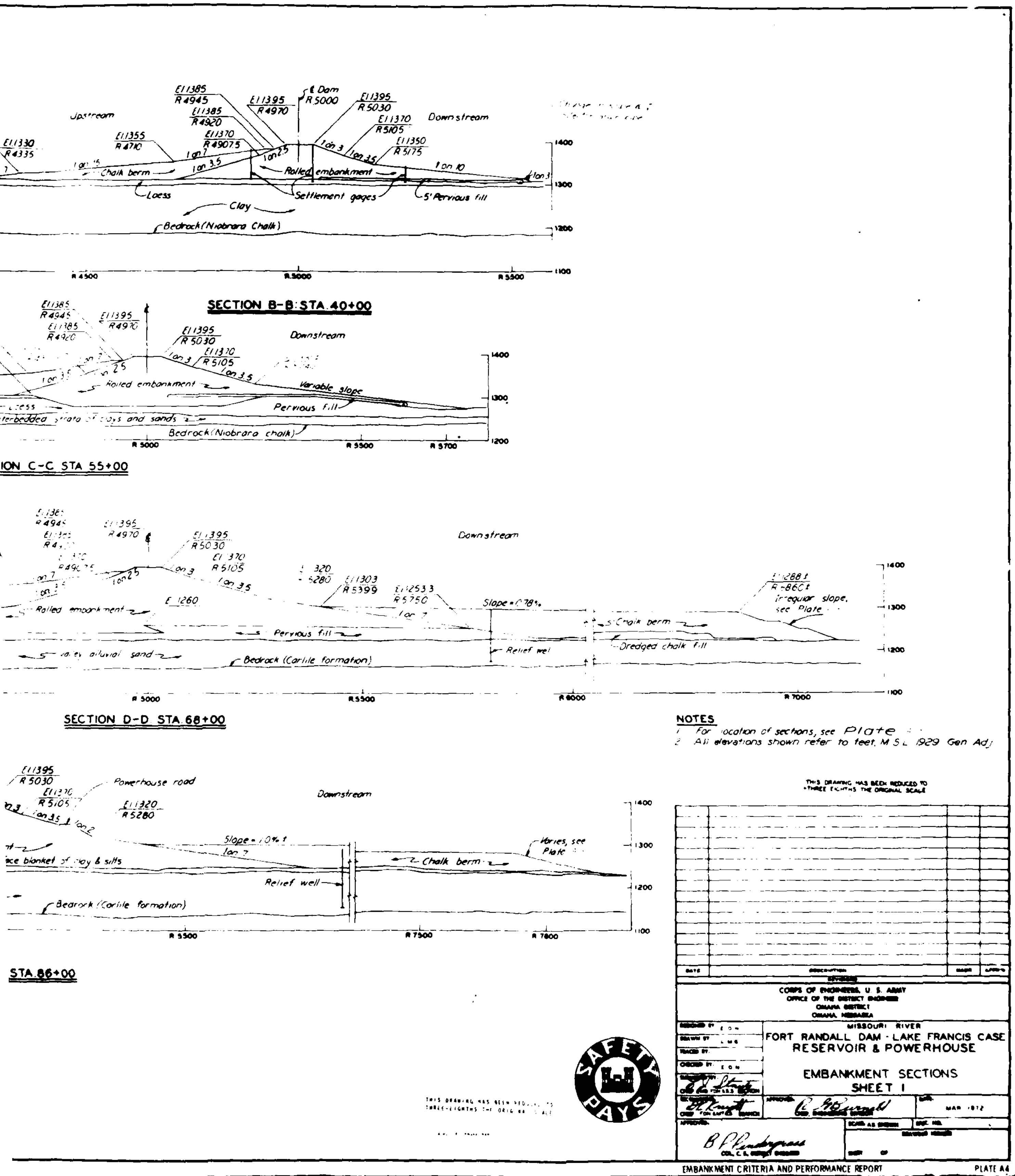
PLATE A3

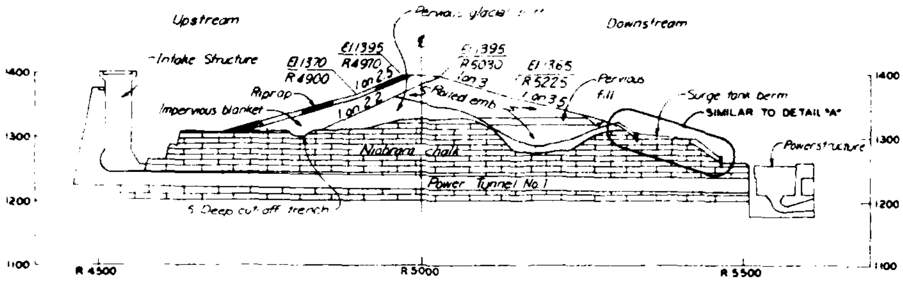
BASE



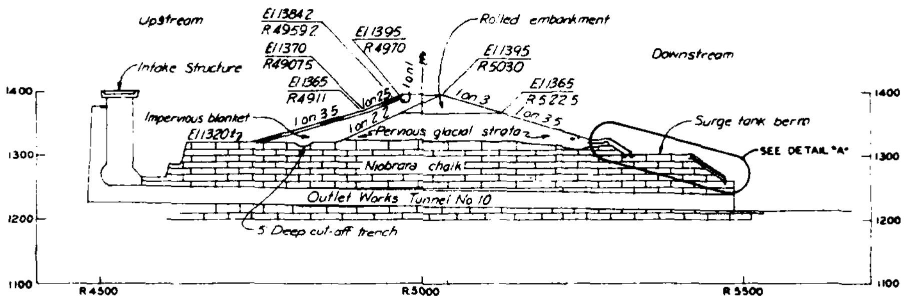
SECTION A-A: STA 20+25
Upstream
Downstream
℄ Dam R 5000
Chalk berm
Clay
Loess
Rolled embankment
Bedrock (Niobrara Chalk)
SECTION B-B: STA 40+00
Upstream
Chalk berm
Rolled embankment
Settlement gages
Loess
Clay
Bedrock (Niobrara Chalk)
SECTION C-C STA 55+00
Upstream
Downstream
Variable, see Plate A-3
Impervious blanket
Chalk berm
Rolled embankment
Loess
Variable slope
Pervious fill
Cutoff trench, see Plate A-3 for location
Interbedded strata of clays and sands
Bedrock (Niobrara chalk)
SECTION D-D STA 68+00
Upstream
Chalk berm
Rolled embankment
Impervious blanket
Pervious fill
Valley alluvial sand
Bedrock (Carlile formation)
SECTION E-E STA 86+00
Upstream
Downstream
Upstream limit of berm varies, see Plate A-3
Chalk berm
Powerhouse road
Rolled embankment
Impervious blanket
Natural surface blanket of clay & silts
Relief well
Valley alluvial sand
Bedrock (Carlile formation)

1

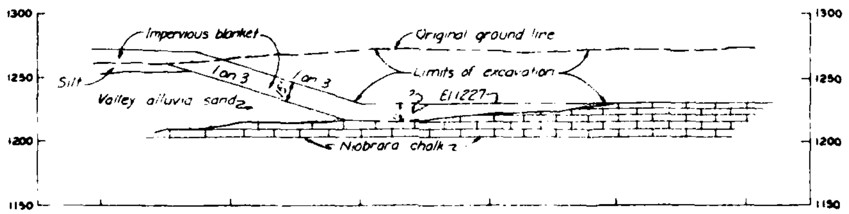

Upstream
Downstream
℄ Dam
El 1385 R 4945
El 1395 R 4970
El 1395 R 5030
El 1385 R 4920
El 1370 R 4907.5
El 1370 R 5105
El 1350 R 5175
El 1355 R 4710
El 1330 R 4335
Chalk berm
Loess
Rolled embankment
Settlement gages
5' Pervious fill
Clay
Bedrock (Niobrara Chalk)
SECTION B-B STA. 40+00
Variable slope
Pervious fill
Interbedded strata of clays and sands
Bedrock (Niobrara chalk)
SECTION C-C STA 55+00
El 1303 R 5399
El 1253.3 R 5750
Slope = 0.78%
Chalk berm
Irregular slope, see Plate
Relief well
Dredged chalk fill
Pervious fill
Bedrock (Carlile formation)
SECTION D-D STA 68+00
Powerhouse road
El 1320 R 5280
Slope = 1.0%
1 on 7
Varies, see Plate
Chalk berm
Relief well
Bedrock (Carlile formation)
STA 86+00
NOTES
1 For location of sections, see Plate
2 All elevations shown refer to feet, M.S.L. 1929 Gen Adj
THIS DRAWING HAS BEEN REDUCED TO THREE EIGHTHS THE ORIGINAL SCALE
CORPS OF ENGINEERS, U. S. ARMY
OFFICE OF THE DISTRICT ENGINEER
OMAHA DISTRICT
OMAHA, NEBRASKA
MISSOURI RIVER
FORT RANDALL DAM - LAKE FRANCIS CASE
RESERVOIR & POWERHOUSE
EMBANKMENT SECTIONS
SHEET 1
MAR 1972
SAFETY PAYS



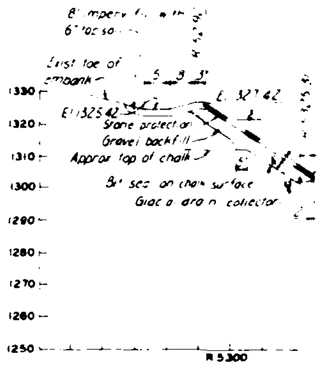
SECTION F-F: STA.107+15

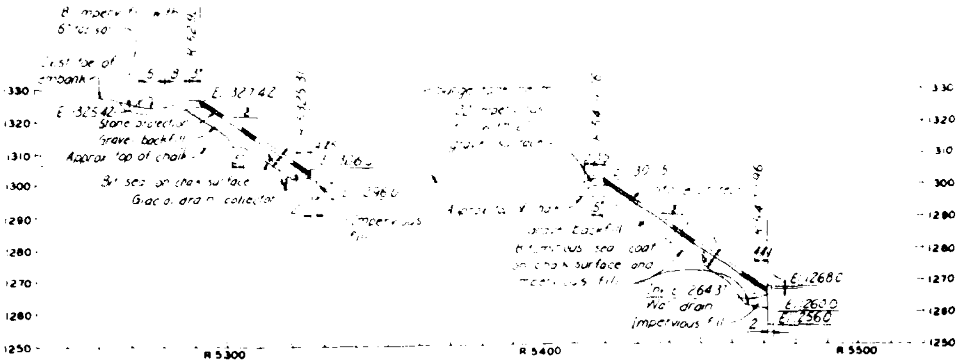


SECTION G-G: STA 113+45



SECTION H-H
APPROACH CHANNEL SLOPE





DETAIL "A"

NOTES

1. For locations of sections, see Plate A3
2. All elevations shown refer to feet, M.S.L., 1929 Gen Adj

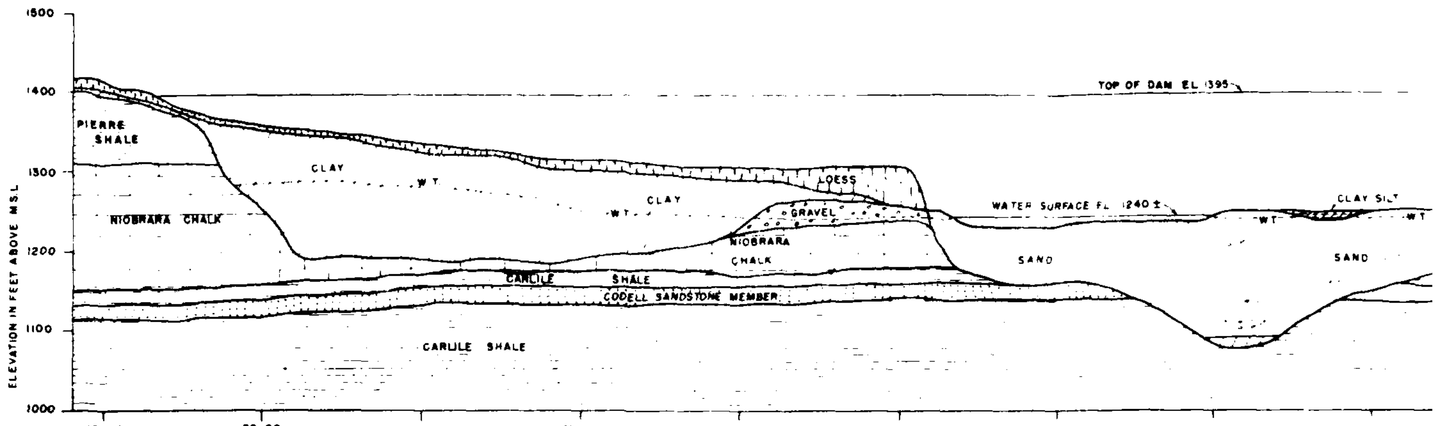
THIS DRAWING HAS BEEN REDUCED TO THREE EIGHTHS THE ORIGINAL SCALE

CORPS OF ENGINEERS, U. S. ARMY
OFFICE OF THE DISTRICT ENGINEER
OMAHA DISTRICT
OMAHA, NEBRASKA

MISSOURI RIVER
FORT RANDALL DAM - LAKE FRANCIS CASE
RESERVOIR & POWERHOUSE

EMBANKMENT SECTIONS
SHEET 2

2

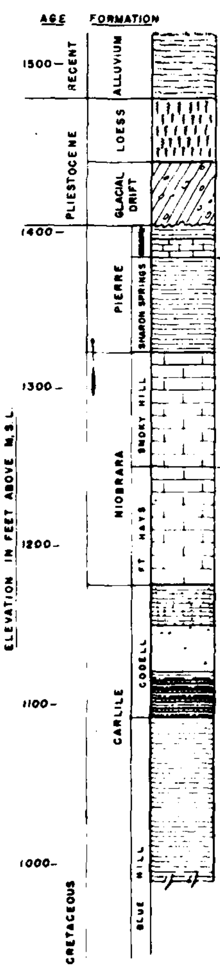


GEOLOGICAL PROFILE ON AXIS OF DAM

SCALE
HORIZ 1" = 400'
VERT 1" = 80'

GEOLOGICAL COLUMN

SCALE 1" = 40'



DESCRIPTION

River sands and alluvial clays. River sands mostly extremely fine sand and silt with few lenses of coarse sand and gravel. Max thickness 105 feet. Alluvial clays are located chiefly on right terrace in an old buried river channel. They have density of 80 lbs/cu.ft., moisture content 20 to 35 % and contain over 60 % clay. Have thickness of over 110 feet.

Wind-borne silt and fine sand, uniform gradation, has low moisture content of 11 % to 16 % and contains 8% to 13 % calcium carbonate. Used for portion of rolled fill embankment on right bank of river. Has average natural density of 80 lbs and maximum compacted density of 105 lbs.

Gravelly clay, sand, gravel and boulders. Thickness up to 70 feet. Over 80% of the glacial drift is till, composed of a gravelly lean clay, the remainder consists of sand, gravel and boulders. Average density 85 lbs per cu.ft. moisture 15 %. Maximum compacted density 118 lbs per cu.ft. The glacial till furnished ideal material for construction of rolled fill embankment at the dam site.

Shale, gray with limestone concretions, only lower few feet present at damsite. Marl: hard dense light gray with small flecks of dark shale scattered throughout. Material not used in foundation of dam. Ranging in thickness from 30 to 80 feet.

Shale dark gray, somewhat bituminous, also contains remains of fish and scales, and numerous bentonite seams ranging from ½ to 3" in thickness. Has average dry density of 108 lbs/cu.ft. and average moisture of 19%. The elevations of the Pierre Shales are such that this material had little bearing on the construction of the Outlet Works. Excavation for the spillway will expose a maximum depth of 80 feet of this material but it does not constitute the foundation for any part of the structure.

Chalk, gray, argillaceous with numerous seams of bentonite clay in upper 30 feet, varying in thickness from ⅛" to 2". In the vicinity of the damsite the Smoky Hill is approximately 70 feet thick. The Smoky Hill has a dry density of 88 lbs per cu.ft., average natural moisture of 22 % and an average compressive strength of 700 P.S.I. The surge tanks are the only major structure in the outlet works to be founded on the Smoky Hill with the exception of the upper portion of the tunnels which were driven thru the Smoky Hill member. A study of the Microfossils by the So. Dakota Geological Survey indicated faunal change between the two members of the Niobrara.

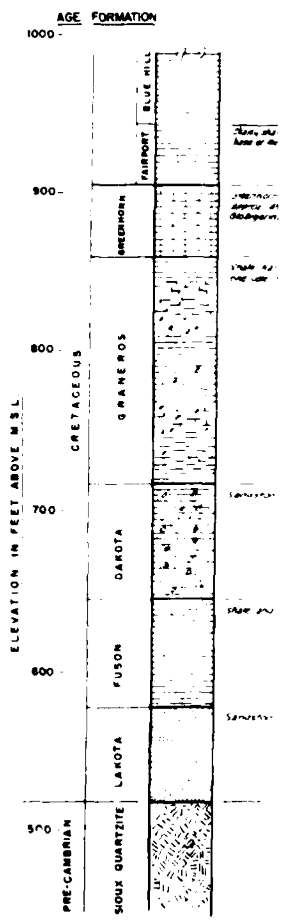
Chalk, light gray, occasionally mottled and banded with darker gray chalk. Contains a 1⅛ marker bentonite approx. 13.0 feet below contact with the Smoky Hill and a few very thin bentonite seams in remaining portion. The Ft. Hays has a dry density of 116 lbs/cu.ft. and a moisture of 18 % and average compressive strength of 1500 P.S.I. The footing for the intake structure, Powerhouse and A, B & C walls were all founded in the Ft. Hays member of the Niobrara chalk.

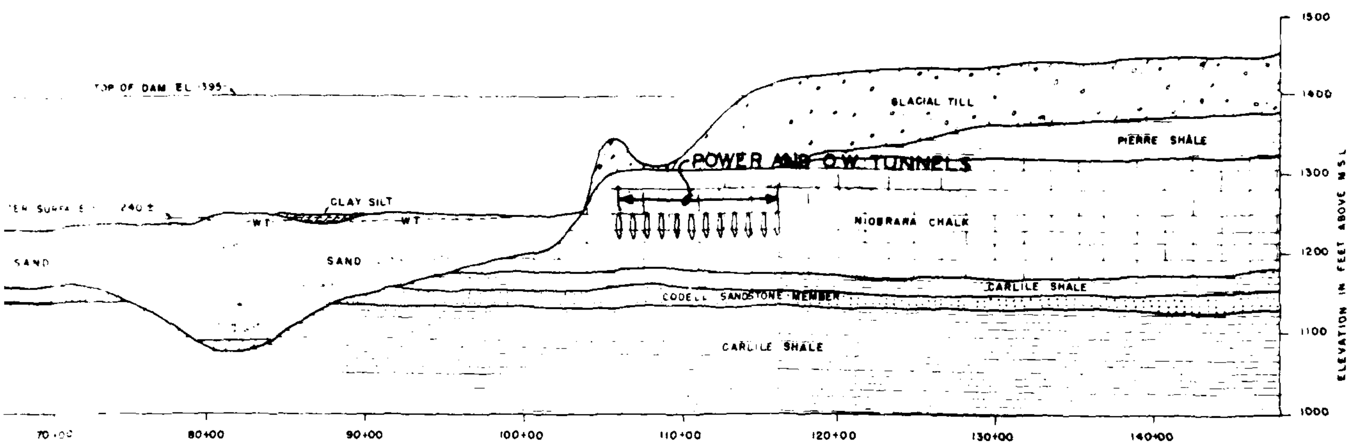
Carlile shale formation is divided into three members. The Codell member is approx. 85 feet thick. It consists of shale and sandy shales and a sandstone layer varying from 20 to 30 feet in thickness. This sandstone layer is known as the Codell sandstone or as the "Tubular sand" by local well drillers. At the damsite it occurs approx. 85 feet below the top of the Carlile formation.

Sandstone, very fine grained, varies in degree of induration from loose to highly cemented, is water bearing and has hydrostatic head of approx. 80 feet at damsite.

Sandy shale with few lenses of sandstone.

Shale, Blue Hill, dark gray and fossiliferous with a few small calcareous concretions. It has a waxy appearance indicating a high clay content.

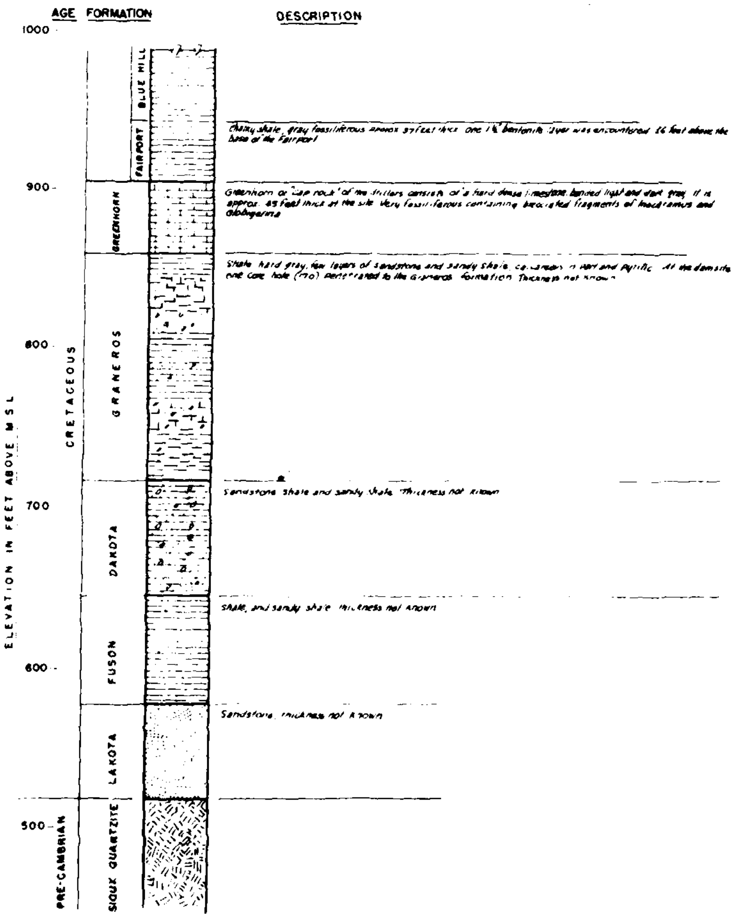




GEOLOGICAL PROFILE ON AXIS OF DAM

SCALE
HORIZ 1"=400'
VERT 1"=80'

GEOLOGICAL COLUMN (CONTINUED)



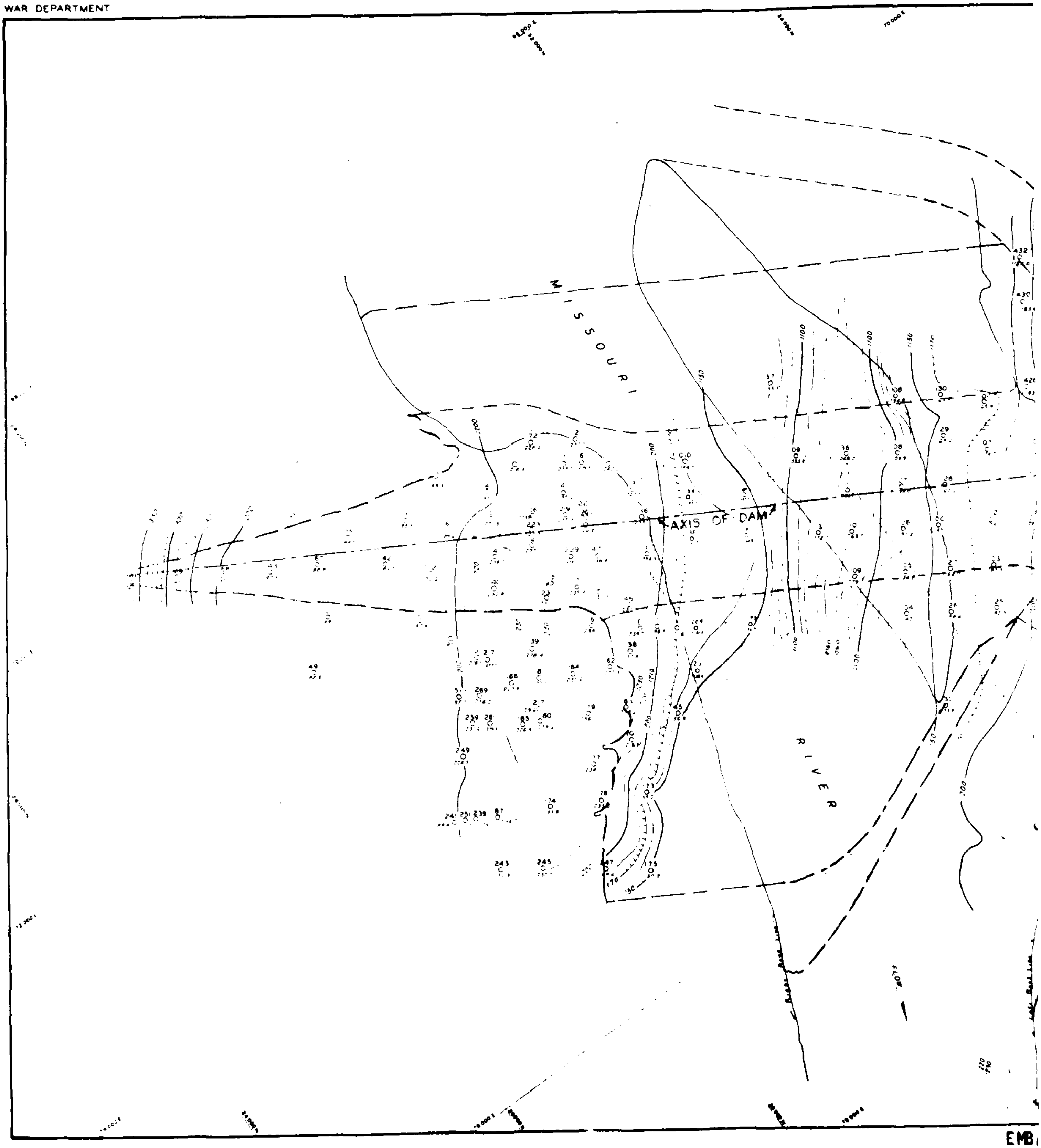
THIS DRAWING HAS BEEN REDUCED TO THREE-EIGHTHS THE ORIGINAL SCALE.

FORT RANDALL RESERVOIR
MISSOURI RIVER BASIN
SOUTH DAKOTA

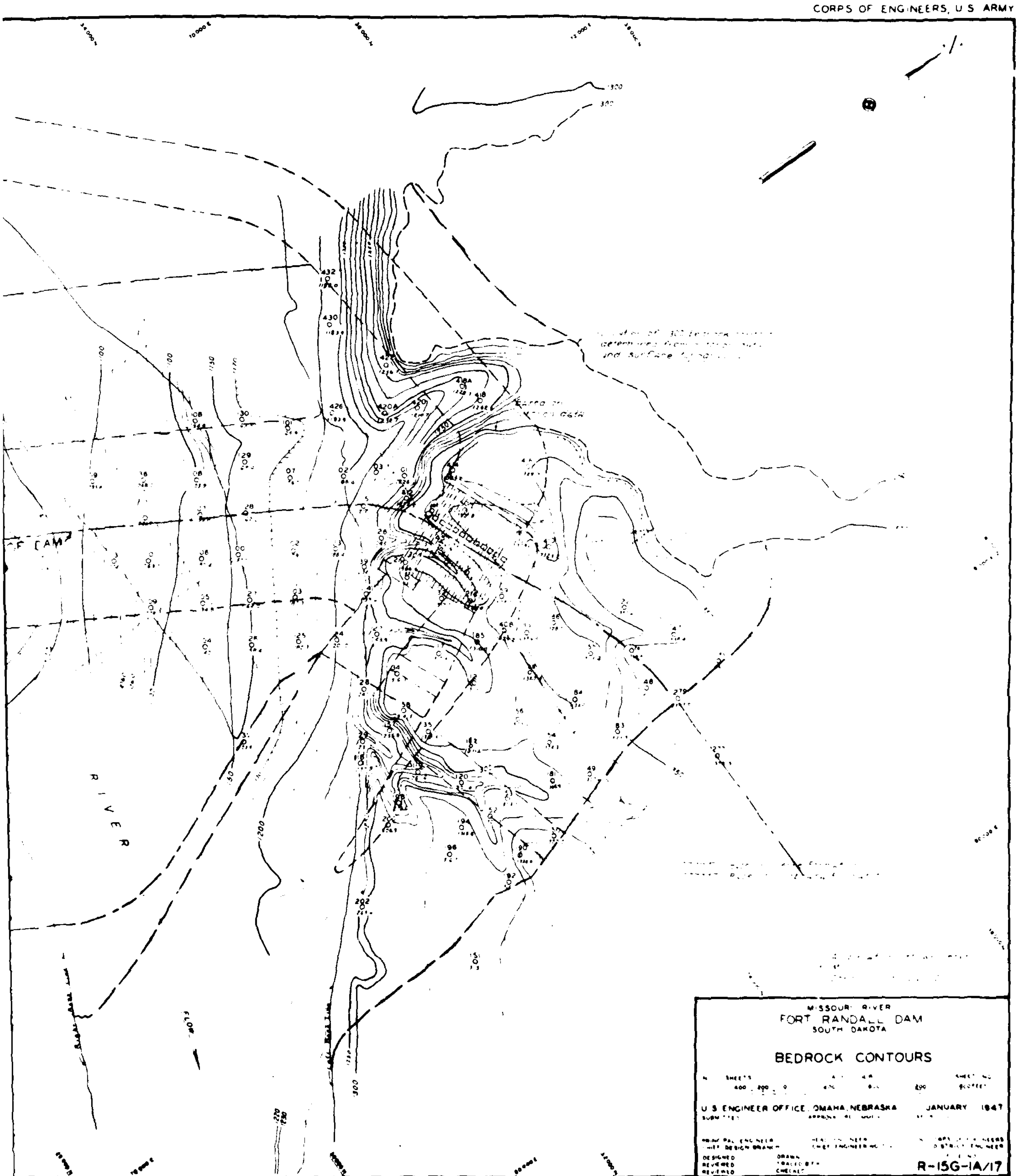
GEOLOGICAL PROFILE ON AXIS OF DAM AND GEOLOGICAL COLUMN

IN SHEETS	SCALE AS SHOWN	SHEET NO.
FORT RANDALL AREA, U.S. ENGINEER OFFICE, PICKSTOWN, SO. DAK.		
SUBMITTED: GEOL. & EXPLORATIONS BRANCH	RECOMMENDED: E.B. Underwood, ENGINEERING DIVISION	APPROVED: AREA ENGINEER
DRAWN BY CEY / TRACED BY CEY	CHECKED BY	DATE MAY 4 1948

WAR DEPARTMENT

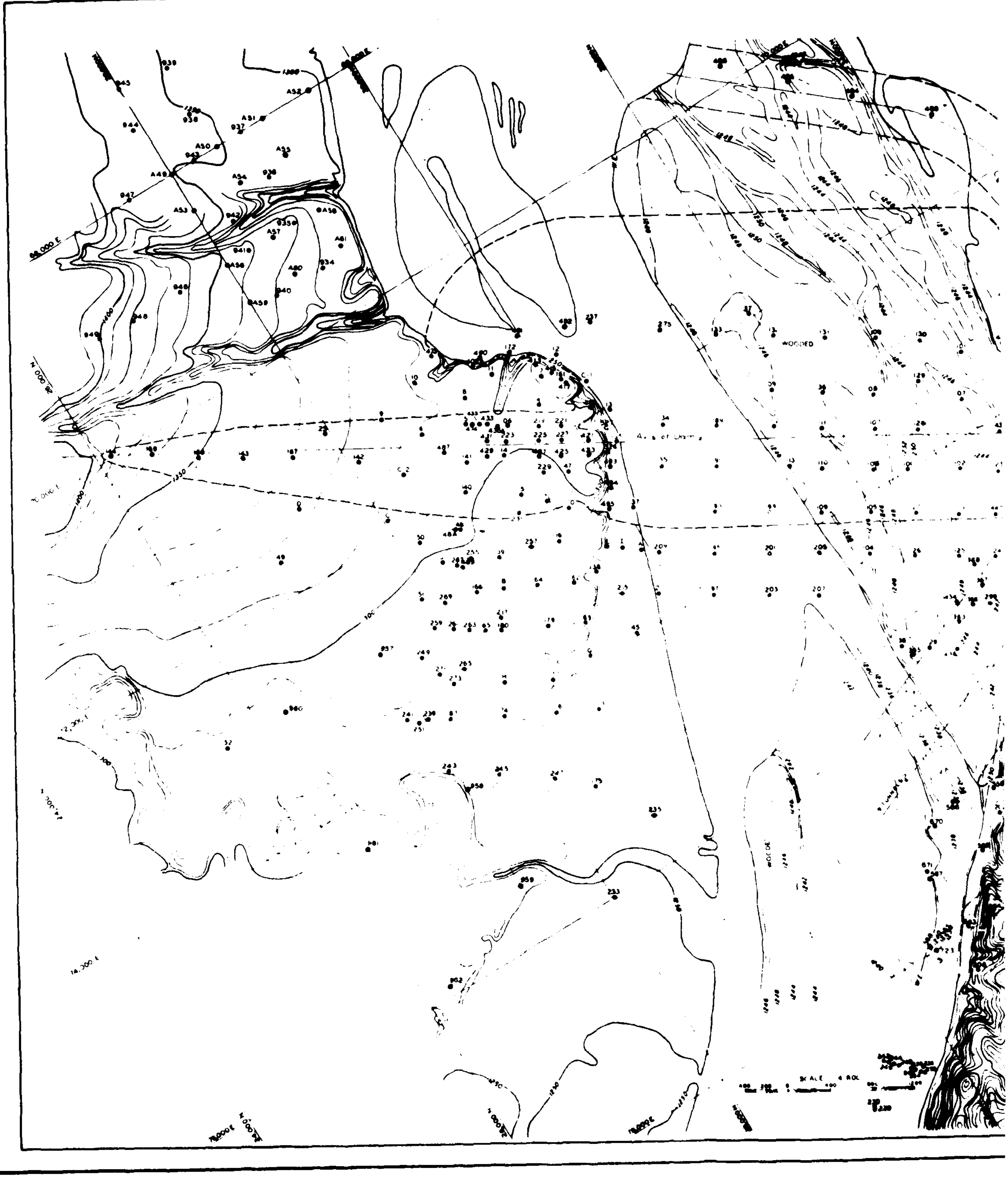


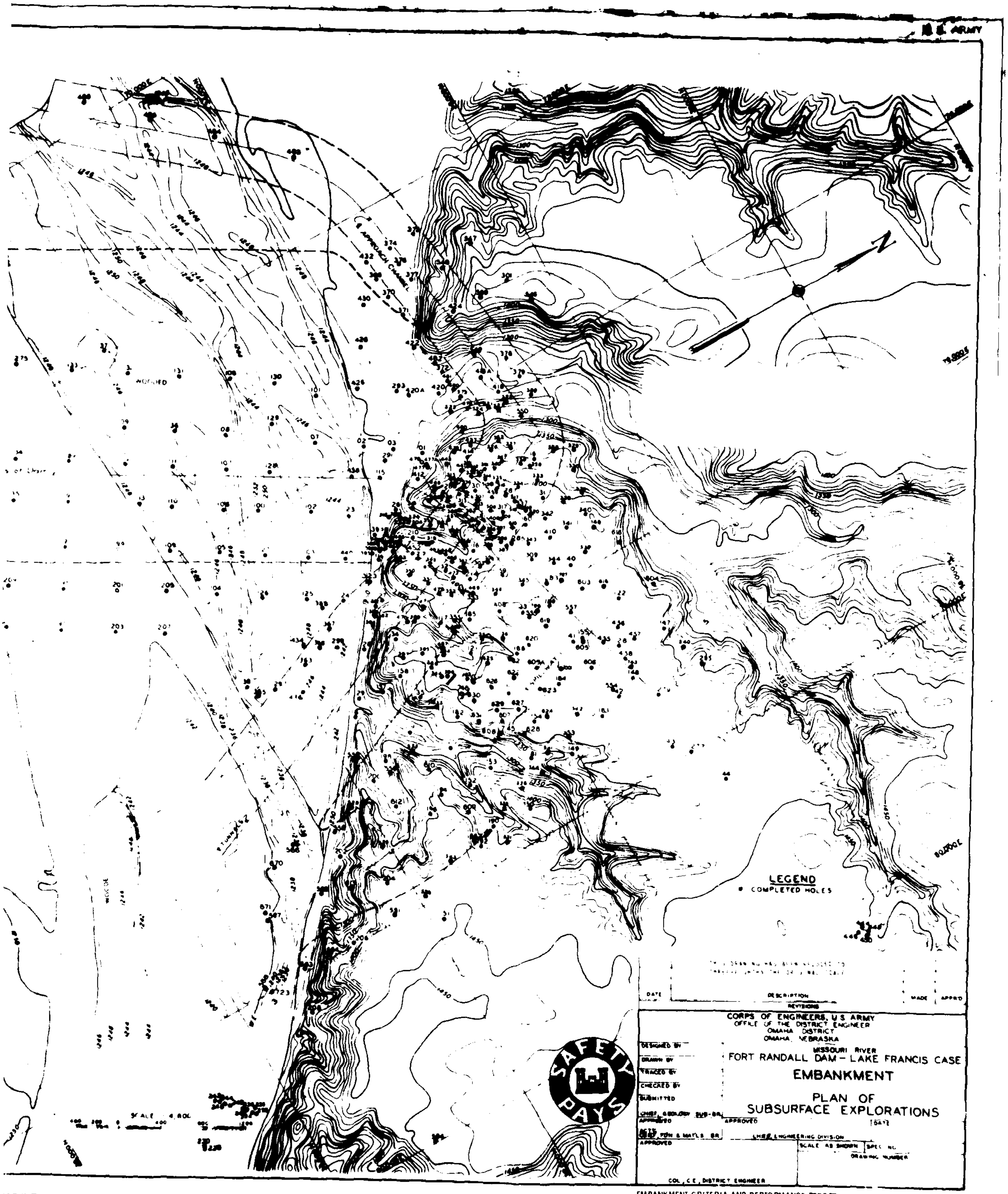
EMB


CORPS OF ENGINEERS, U.S. ARMY
RIVER
MISSOURI RIVER
FORT RANDALL DAM
SOUTH DAKOTA
BEDROCK CONTOURS
U.S. ENGINEER OFFICE, OMAHA, NEBRASKA
JANUARY 1947
R-15G-1A/17

2

CORPS OF ENGINEERS




U.S. ARMY
LEGEND
COMPLETED HOLES
DATE
DESCRIPTION
MADE
APPR'D
REVISIONS
CORPS OF ENGINEERS, U.S. ARMY
OFFICE OF THE DISTRICT ENGINEER
OMAHA DISTRICT
OMAHA, NEBRASKA
DESIGNED BY
DRAWN BY
TRACED BY
CHECKED BY
SUBMITTED
MISSOURI RIVER
FORT RANDALL DAM – LAKE FRANCIS CASE
EMBANKMENT
PLAN OF
SUBSURFACE EXPLORATIONS
SAFETY PAYS
APPROVED
DATE
CHIEF, ENGINEERING DIVISION
APPROVED
SCALE AS SHOWN
SPEC. NO.
DRAWING NUMBER
COL., C.E., DISTRICT ENGINEER

PIEZOMETER TUBE AND OBSERVATION WELL DATA

Group	Piezometer or Well Number	Location Station	Location Range	Elevation Top of Pipe	Elevation Bottom of Screen	Type	Purpose of Piezometer Tube or Observation Well
	A5.2	48+00	5200	1350.06	1303.04		To record seepage pressures in the downstream pervious drain.
	A5.5	48+00	5500	1318.38	1300.58		To record seepage pressures in terrace substrata and to determine the effectiveness of the upstream cutoff trench.
	B4.95	56+00	4950	1388.62	1242.19		
	B5.2	56+00	5200	1345.49	1291.49		To record seepage pressures in the downstream pervious drain.
	B5.26	55+98	5259	1329.88	1298.18		To record seepage pressures in terrace substrata and to determine the effectiveness of the upstream cutoff trench.
	B5.5	56+00	5500	1303.02	1247.9		
	C4.95	70+00	4950	1389.33	1224.09		To record underseepage pressures in the valley alluvial sands.
	C5.2	70+00	5200	1345.19	1224.29		
	C5.45	70+00	5450	1305.04	1224.64		
	C5.75	70+00	5750	1302.70	1224.60		
	D4.95	81+50	4950	1384.78	1224.68		
	D5.2	81+50	5200	1343.56	1221.56		
	D5.45	81+50	5450	1307.62	1219.32	Well Point	
1	D5.75	81+50	5750	1301.78	1213.0		
	D6.2	81+50	6200	1297.66	1214.96		
	E4.95	95+00	4950	1387.22	1219.52		
	E5.2	95+00	5200	1343.70	1225.70		
	E5.45	95+00	5450	1304.75	1223.55		
	E5.75	95+00	5750	1285.68	1226.88		
	F4.9	109+25	4900	1368.10	1318.10		To record seepage pressures in the pervious strata overlaying the chalk formation.
	F4.96	109+25	4960	1396.29	1329.49		
	F5.04	109+25	5040	1395.66	1308.88		To record seepage pressures in the downstream pervious drain.
	F5.12	109+25	5217	1338.90	1312.90		
	H5.038	108+90	5030	1396.46	1229.86		To record the hydrostatic pressures that develop in the chalk formation between tunnels 3 & 4.
	I5.038	112+35	5035	1396.34	1230.14	Casagrande	To record the hydrostatic pressures that develop in the chalk formation between tunnels 8 & 9.
	K5.035	114+50	5035	1397.44	1229.34		To record the hydrostatic pressures that develop in the chalk formation between tunnels 11 & 12.
	L5.21	120+50	5210	1351.19	1323.79	Well Point	To check the efficiency of the pervious drain adjacent to the right spillway wall at the downstream toe of the embankment.
	C6.829	70+00	6829	1290.34	1218.84		To record the hydrostatic pressure in the valley alluvium at the toe of the chalk berm.
	M6.801	75+31	6801	1289.49	1218.19		
	M6.796	75+34	6796	1290.15	1231.95		To record the hydrostatic pressure at the downstream toe of the chalk berm.
	PZ-1	61+30	5800	1302.13	1237.73		
	PZ-2	63+30	5800	1301.24	1228.24		
	PZ-3	66+30	5800	1302.02	1225.02		
	PZ-4	69+30	5600	1301.24	1224.74		
	PZ-4A	69+30	5805	1300.96	1186.96		
	PZ-5	72+30	5800	1301.49	1221.19		
	PZ-6	75+15	5800	1303.18	1225.28		
3	PZ-7	77+35	5850	1302.02	1223.52	Well Point	To record the uplift pressures in the valley alluvium along the line of relief wells.
	PZ-8	79+35	5850	1301.92	1219.32		
	PZ-9	82+30	5875	1301.84	1224.64		
	PZ-10	84+70	5875	1301.58	1229.58		
	PZ-11	87+60	5875	1300.70	1229.70		
	PZ-12	90+60	5900	1279.95	1234.95		
	PZ-13	94+10	5900	1277.26	1235.26		
	PZ-14	98+03	5900	1274.78	1234.78		
		Outlet Works Grid System					
	B	258+98	275	1272.85	1135.9	(Temporary Obs. Wells)	
	C	263+04	215	1270.70	1135.0		
	G	257+79	1350	1324.02	1133.7		
	B Wall	B Wall Outlet Works		1253.7	1135.7	Obs. Well	
5	P-1	Spillway Pier 1		1353.5	1125	(Perm. Relief Wells)	To record the artesian pressures in the Codell sandstone.
	P-11	Spillway Pier 11		1353.5	1125		
	P-20	Spillway Pier 20		1353.5	1125		
	SP-1	Block 62 Rt. Spillway Wall		1258.88	1150	(Piezometer Wells)	
	SP-2	Block 62 Lt. Spillway Wall		1258.88	1150		

NOTE:

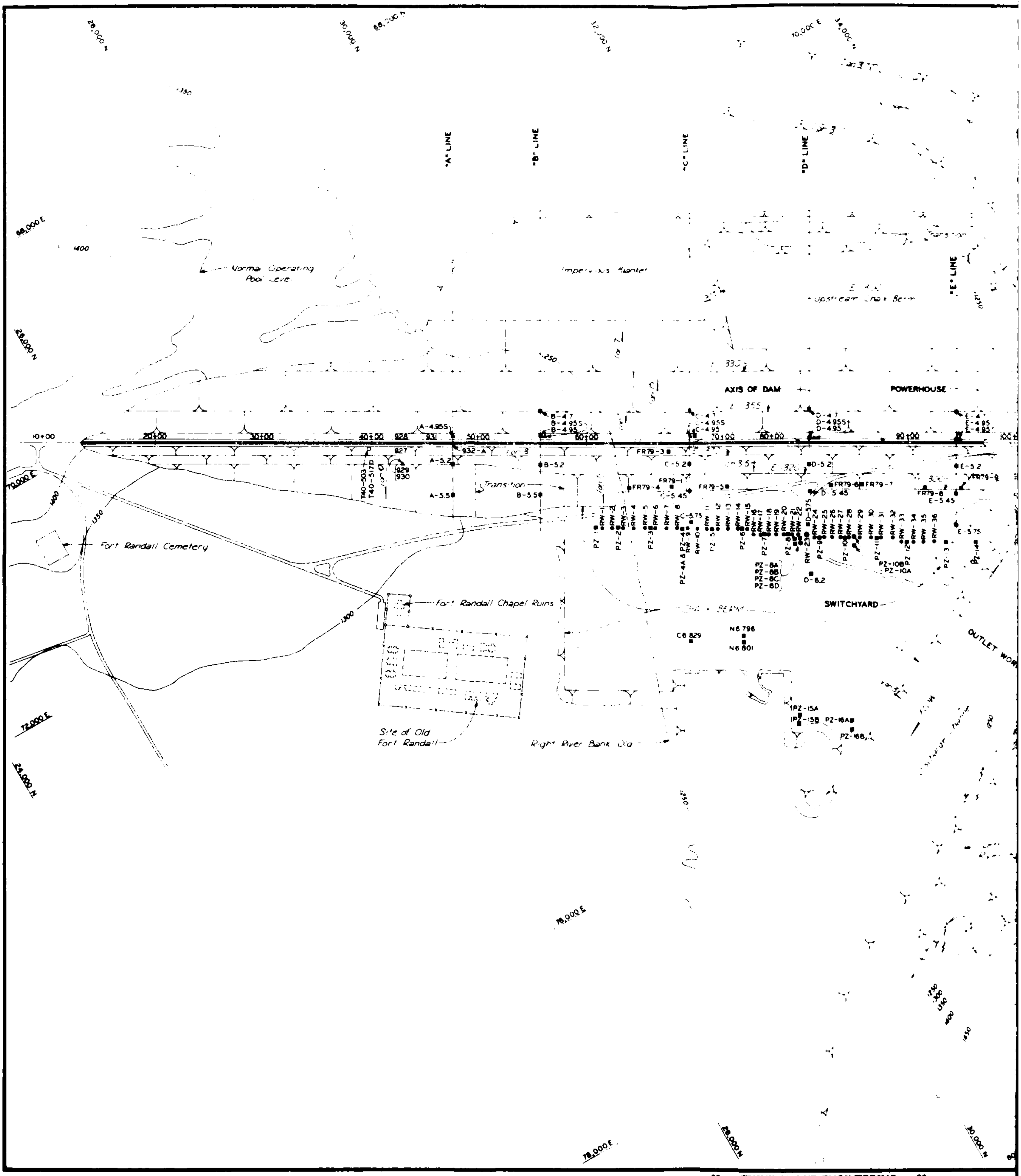
1. The elevation of top of pipe as given above is the most recent elevation available at the time of revision of this table.

2. Piezometer tubes in group 1 through 4 are located by the Embankment Grid System while group 5 observation wells are located by the Outlet Works Grid System.

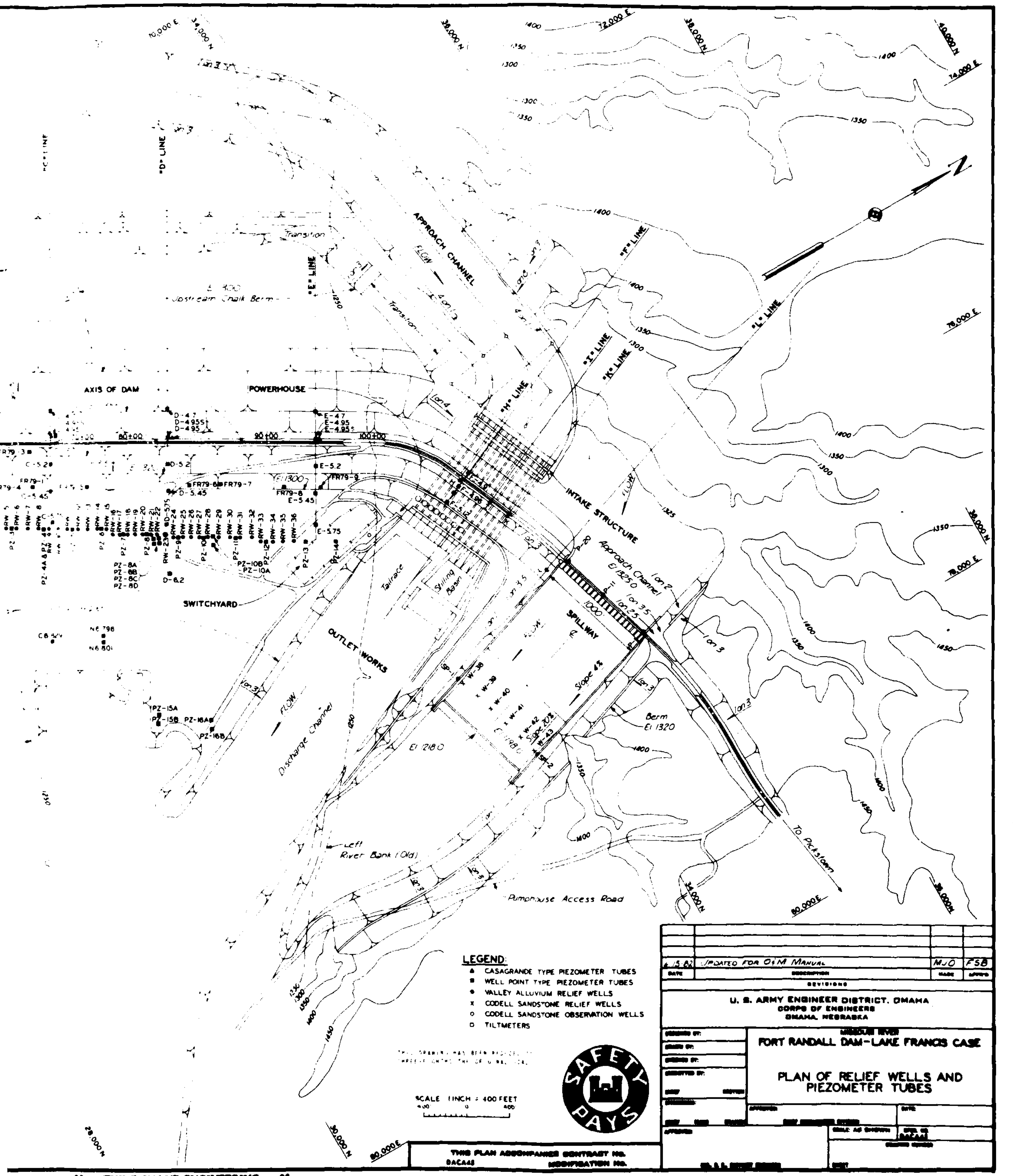
(Additional Piezometers Since 1976)
Piezometer Tube and Observation Well Data

Group	Piezometer or Well Number	Location: Station	Location: Range	Elevation: Top of Pipe	Elevation: Bottom of Screen	Type	Purpose of Piezometer Tube or Observation Well
2	FR79-1	68+00	5425	1302.35	1206.35	Well Point	To record uplift pressure in the valley alluvium upstream but parallel to the line of embankment relief wells.
	FR79-3	68+00	5030	1396.80	1183.60		
	FR79-4	64+00	5425	1302.85	1182.05		
	FR79-5	74+00	5425	1304.50	1137.15		
	FR79-6	80+00	5425	1307.00	1173.00		
	FR79-7	86+00	5425	1306.50	1132.50		
	FR79-8	92+00	5445	1309.80	1174.10		
	FR79-9	96+00	5445	1302.10	1184.60		
3	PZ-8A	80+00	5850	1302.05	1121.85	Well Point	To record hydrostatic pressures and water gradient in the valley alluvial sand. PZ-8A and PZ-8B serve as checks on PZ-8C and PZ-8D.
	PZ-8B	79+90	5850	1302.25	1181.85		
	PZ-8C	79+80	5850	1302.15	1120.90 *		
	PZ-8D	79+8	5850	1302.30	1181.75		
	PZ-10A	85+00	5875	1301.70	1135.35 *		
	PZ-10B	85+00	5875	1302.05	1180.65		
4	PZ-15A	80+00	7400	1290.25	1114.70 *	Well Point	To record hydrostatic pressures and water gradient in the downstream valley alluvial sand.
	PZ-15B	80+00	7400	1290.20	1164.25		
	PZ-16A	85+00	7400	1278.60	1131.07 *		
	PZ-16B	85+00	7400	1278.75	1176.60		
5	PP-1	255+04	3+85.20	1260.15	1230.15	Well Point	To determine the water seepage level in the gravel backfill beneath the tunnel terminal
	PP-2	254+17	3+77.17	1260.50	1245.50		
	PF-3	254+86	3+57.20	1262.40	1247.40		

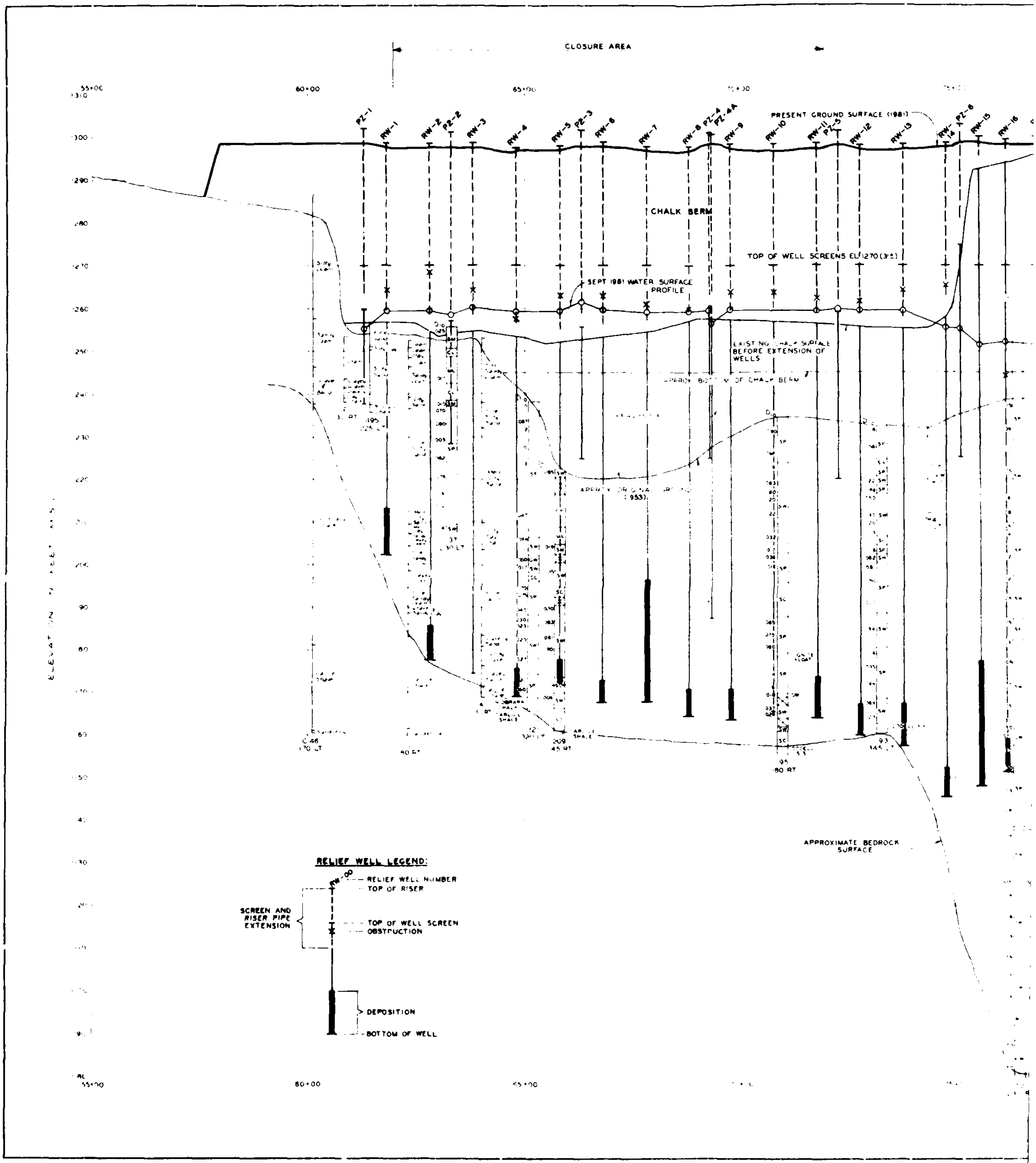
NOTE: *These piezometers are installed into same drill hole.
Also see notes on previous page.

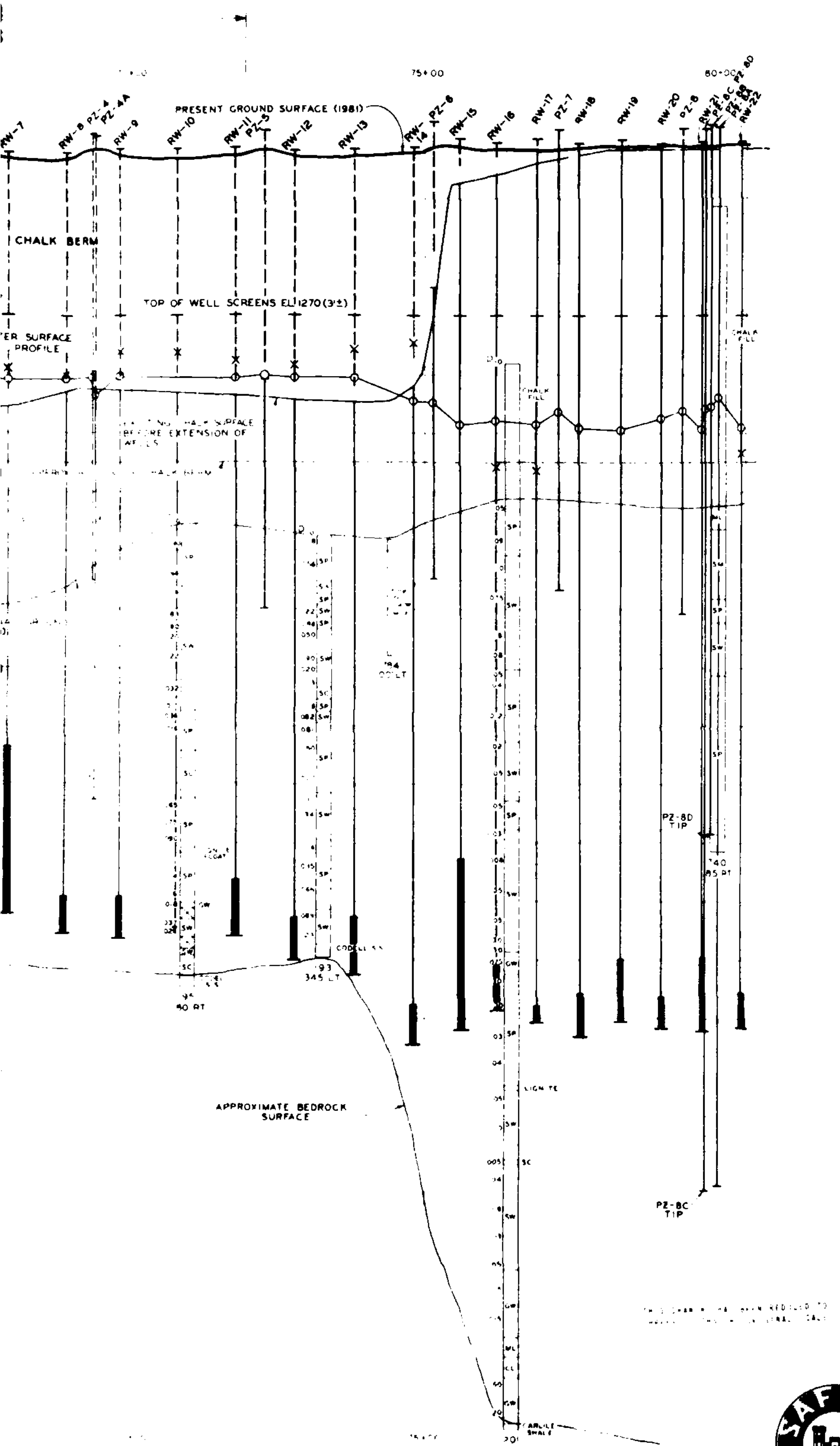


"A" LINE
"B" LINE
"C" LINE
"D" LINE
"E" LINE
Normal Operating Pool Level
Impervious Blanket
Upstream Chalk Berm
AXIS OF DAM
POWERHOUSE
Transition
Fort Randall Cemetery
Fort Randall Chapel Ruins
Site of Old Fort Randall
SWITCHYARD



$$ – THINK VALUE ENGINEERING – $$


CLOSURE AREA
55+00
60+00
65+00
PRESENT GROUND SURFACE (1981)
PZ-1
RW-1
RW-2
PZ-2
RW-3
RW-4
RW-5
PZ-3
RW-6
RW-7
RW-8
PZ-4
PZ-4A
RW-9
RW-10
RW-11
PZ-5
RW-12
RW-13
RW-14
PZ-6
RW-15
RW-16
CHALK BERM
TOP OF WELL SCREENS EL 1270(3±)
SEPT 1981 WATER SURFACE PROFILE
EXISTING CHALK SURFACE BEFORE EXTENSION OF WELLS
APPROXIMATE BEDROCK SURFACE
RELIEF WELL LEGEND:
RELIEF WELL NUMBER
TOP OF RISER
SCREEN AND RISER PIPE EXTENSION
TOP OF WELL SCREEN
OBSTRUCTION
DEPOSITION
BOTTOM OF WELL



PRESSURE RELIEF WELL LOCATIONS				
RELIEF WELL NO.	STATION	RANGE	BOTTOM ELEVATION	TOP RISER ELEVATION
1	61+80	5800	1202.48	1298.48
2	62+80	5800	1177.70	1298.70
3	63+80	5800	1171.81	1298.81
4	64+80	5800	1169.27	1297.87
5	65+80	5800	1171.91	1298.11
6	66+80	5800	[illegible]	1299.18
7	67+80	5800	1167.83	1298.03
8	68+80	5800	1164.26	1298.06
9	69+80	5800	[illegible]	1298.60
10	70+80	5800	[illegible]	1298.55
11	71+80	5800	[illegible]	1298.70
12	72+80	5800	[illegible]	1298.08
13	73+80	5800	1157.62	1298.42
14	74+80	5800	1145.86	1298.86
15	75+60	5800	1148.00	1299.80
16	76+20	5850	1151.33	1299.43
17	76+90	5850	1149.34	1299.94
18	77+60	5850	1146.98	1299.48
19	78+30	5830	1149.64	1299.94
20	79+00	5850	1148.44	1299.34
21	79+70	5850	1148.02	1299.32
22	80+40	5850	1148.83	1299.63

PIEZOMETER TUBE LOCATIONS				
PIEZ TUBE NO.	STATION	RANGE	BOTTOM ELEVATION	TOP TUBE ELEVATION
1	61+30	5800	1237.64	1302.04
2	63+30	5800	1228.42	1301.12
3	66+30	5800	1225.10	1301.90
4	69+30	5800	1224.30	1301.10
4A	69+30	5805	1187.03	1300.83
5	72+30	5800	1220.84	1301.34
6	75+15	5800	1225.10	1303.00
7	77+35	5850	1223.35	1301.85
8	79+35	5850	1219.16	1301.76
8A	80+00	5850	1121.85	1302.05
8B	79+90	5850	1121.85	1302.25
8C	79+80	5850	1120.90	1302.15
8D	79+80	5850	1181.75	1302.30

DRILL HOLE LEGEND:

D_{10}
000
000

GW	Gravel or Sandy Gravel, Well-Graded
GP	Gravel or Sandy Gravel, Poorly-Graded
GM	Silty Gravel or Silty Sandy Gravel
GC	Clayey Gravel or Clayey Sandy Gravel
SW	Sand or Gravelly Sand, Well-Graded
SP	Sand or Gravelly Sand, Poorly-Graded
SM	Silty Sand or Silty Gravelly Sand
SC	Clayey Sand or Clayey Gravelly Sand
ML	Silts, Sandy Silts, Gravelly Silts or Diatomaceous Soils
CL	Lean Clays, Sandy Clays or Gravelly Clays
OL	Organic Silts or Lean Organic Clays
MH	Micaceous Clays or Diatomaceous Soils
CH	Fat Clays
OH	Fat Organic Clays
000	HOLE NUMBER
000	LT. OR RT. OFFSET FROM RANGE OF WELLS

NOTES:

1. Where laboratory classifications of samples have been made using the "Uniform Soils Classification" it is designated on the logs by such letters as ML, SW, GP, etc., otherwise field indentification of materials by inspectors is used and designated as such by a description of the material.
2. The values on the left side of the log are the D_{10} effective size.
3. "PZ" refers to Piezometer.
4. PZ-8A, 8B, 8C and 8D were installed after 1976.
5. Relief well and piezometer elevations shown are based on 1981 data.

DATE | DESCRIPTION | MADE | APPR

REVISIONS

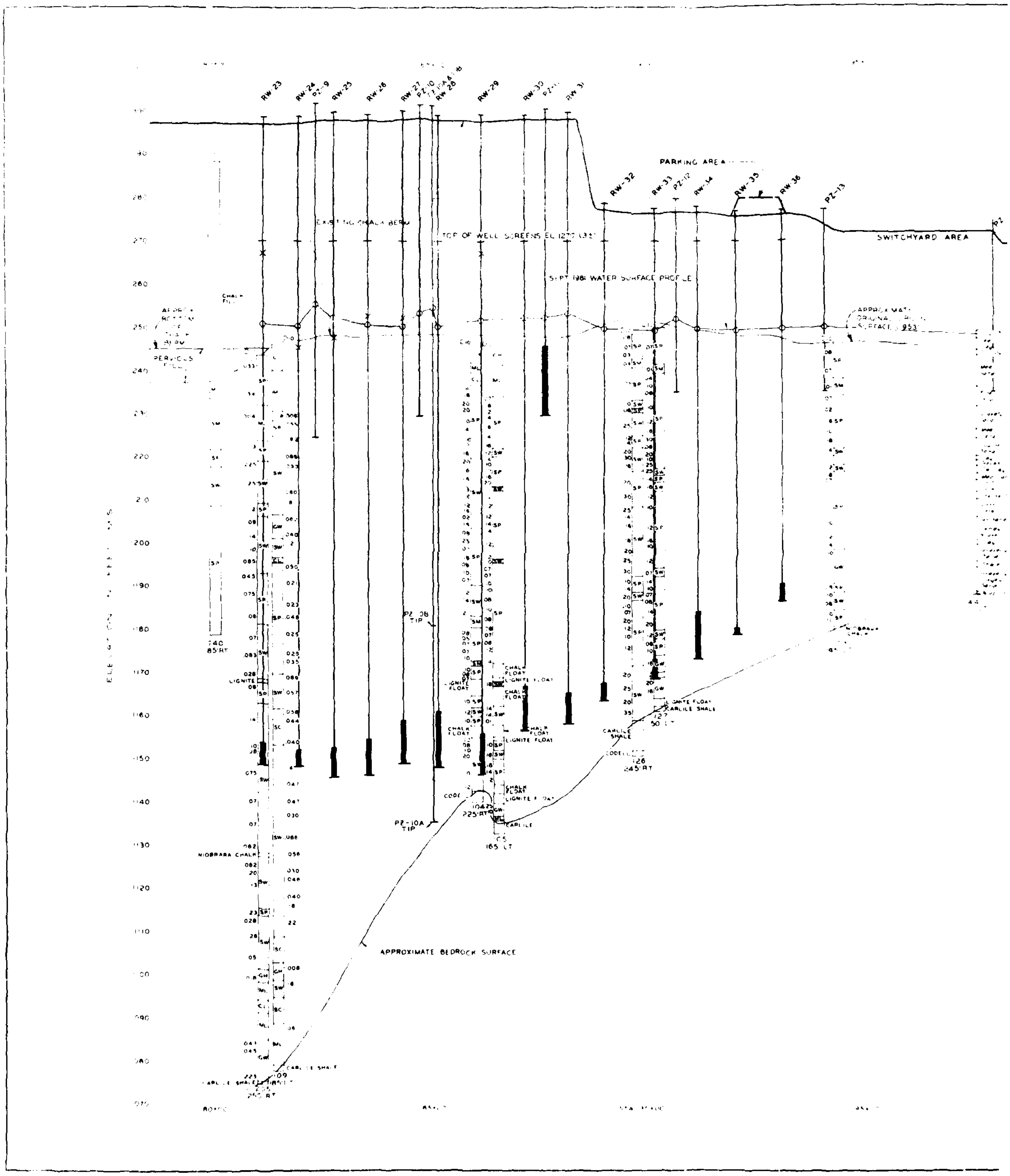
CORPS OF ENGINEERS, U. S. ARMY
OFFICE OF THE DISTRICT ENGINEER
OMAHA DISTRICT
OMAHA, NEBRASKA

DESIGNED BY
DRAWN BY
TRACED BY
CHECKED BY
SUBMITTED BY

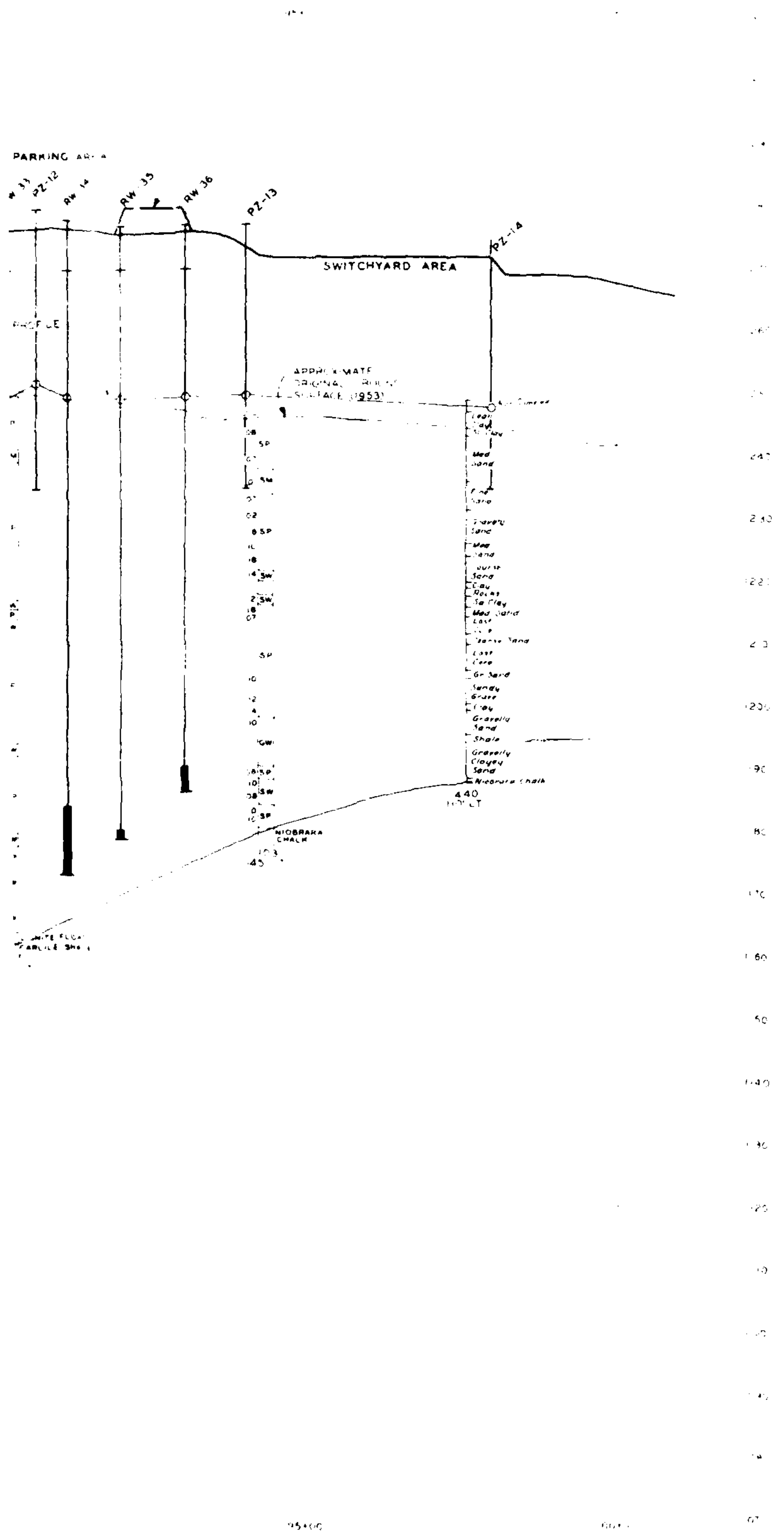
MISSOURI RIVER
FORT RANDALL DAM - LAKE FRANCIS CASE

RELIEF WELLS & PIEZOMETER TUBES
SOILS PROFILE SHEET 1

APPROVED | DATE
CHIEF, ENGINEERING DIVISION
APPROVED
COL. C. E. DISTRICT ENGINEER
SCALE AS SHOWN | SPEC NO.
DRAWING NUMBER



RW-23
RW-24
PZ-9
RW-25
RW-26
RW-27
PZ-10
RW-28
RW-29
RW-30
RW-31
PARKING AREA
RW-32
RW-33
PZ-12
RW-34
RW-35
RW-36
PZ-13
SWITCHYARD AREA
EXISTING CHALK BERM
TOP OF WELL SCREENS
WATER SURFACE PROFILE
PERVIOUS FILL
NIOBRARA CHALK
LIGNITE
LIGNITE FLOAT
CHALK FLOAT
CARLILE SHALE
PZ-10B TIP
PZ-10A TIP
APPROXIMATE BEDROCK SURFACE
165' LT
225' RT
245' RT



NOTES:

1. PZ-10A and 10B were installed after 1976.
2. PZ-11 has 15.90 feet of deposition as of 1981. Depositions in all other piezometers are negligible
3. For relief well legend refer to Plate A-

DATE	DESCRIPTION
	REVISIONS

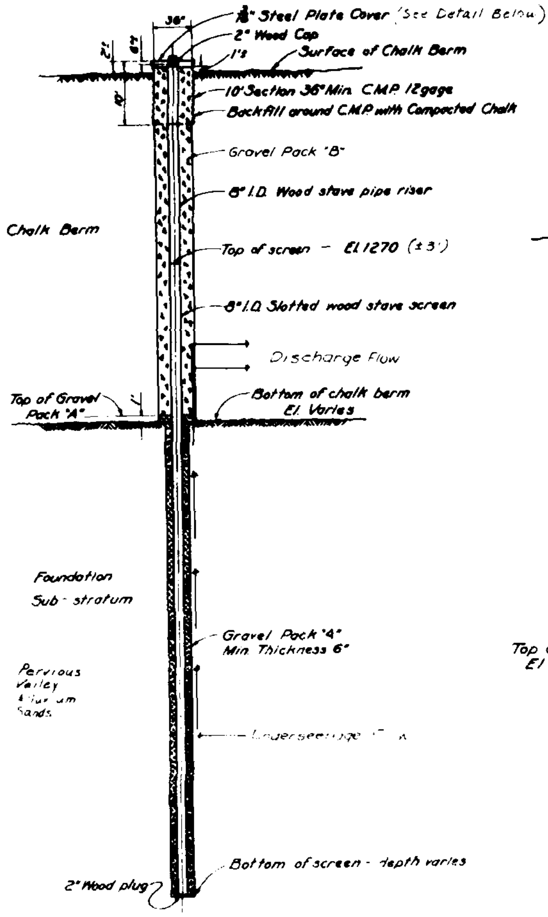
CORPS OF ENGINEERS U. S. ARMY
OFFICE OF THE DISTRICT ENGINEER
OMAHA DISTRICT
OMAHA, NEBRASKA

DESIGNED BY
DRAWN BY
TRACED BY
CHECKED BY
SUBMITTED BY
CHIEF,
RECOMMEND
CHIEF,
APPROVED

MISSOURI RIVER
FORT RANDALL DAM - LAKE FRANCIS CASE
RELIEF WELLS & PIEZOMETER TUBES
SOILS PROFILE SHEET 2

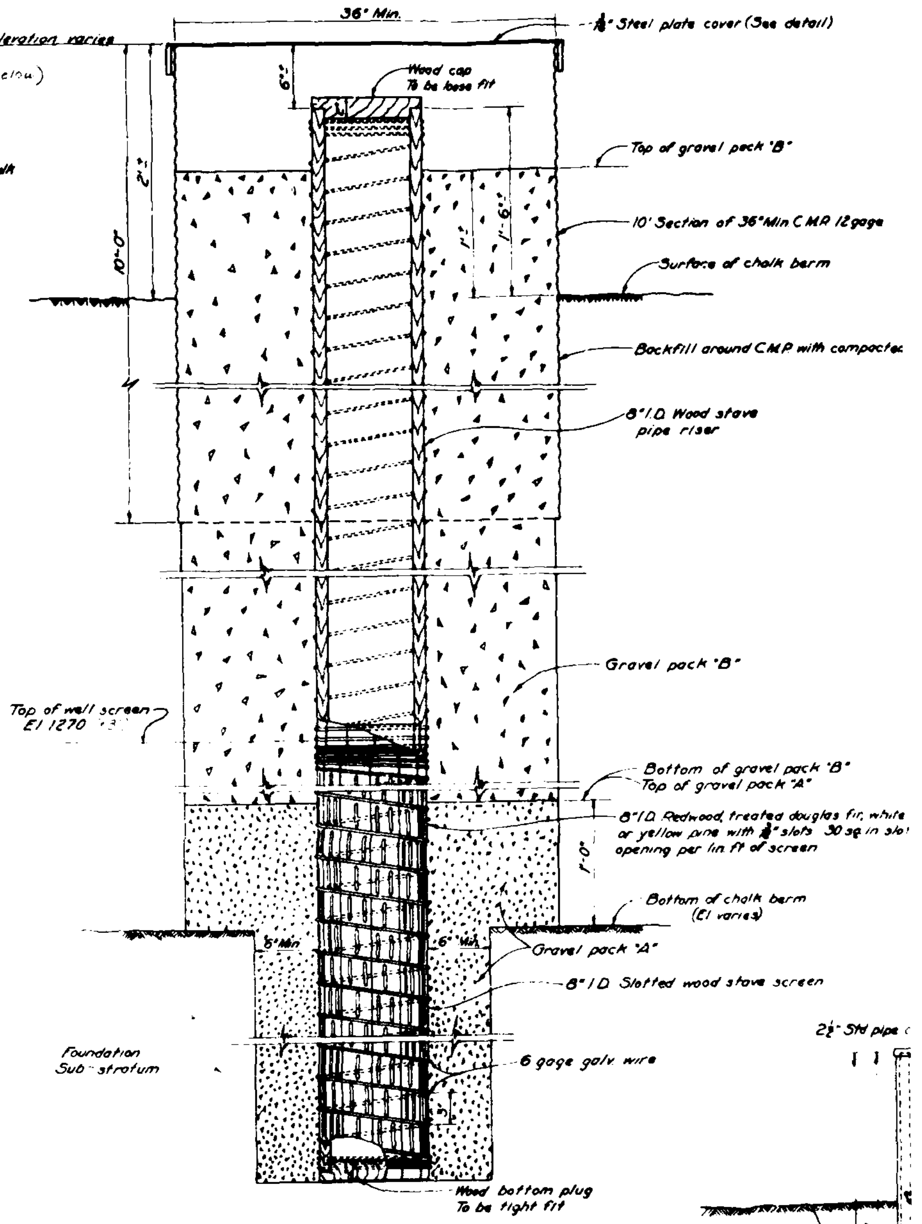
MAR 1972

2



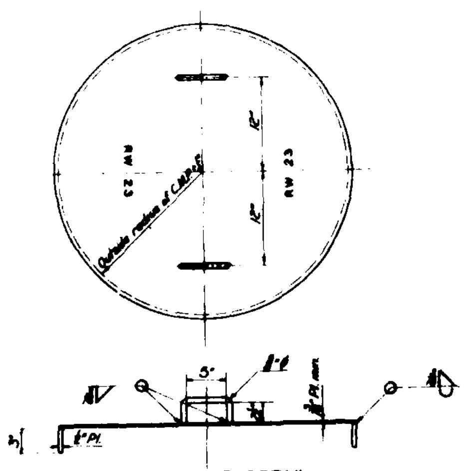
TYPICAL PRESSURE RELIEF WELL

NO SCALE



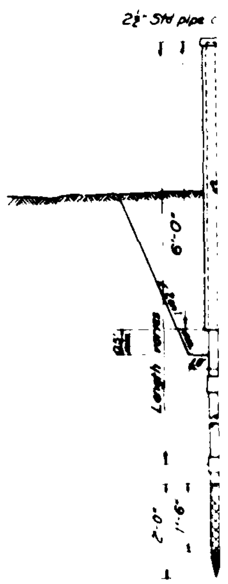
RELIEF WELL DETAILS

NO SCALE



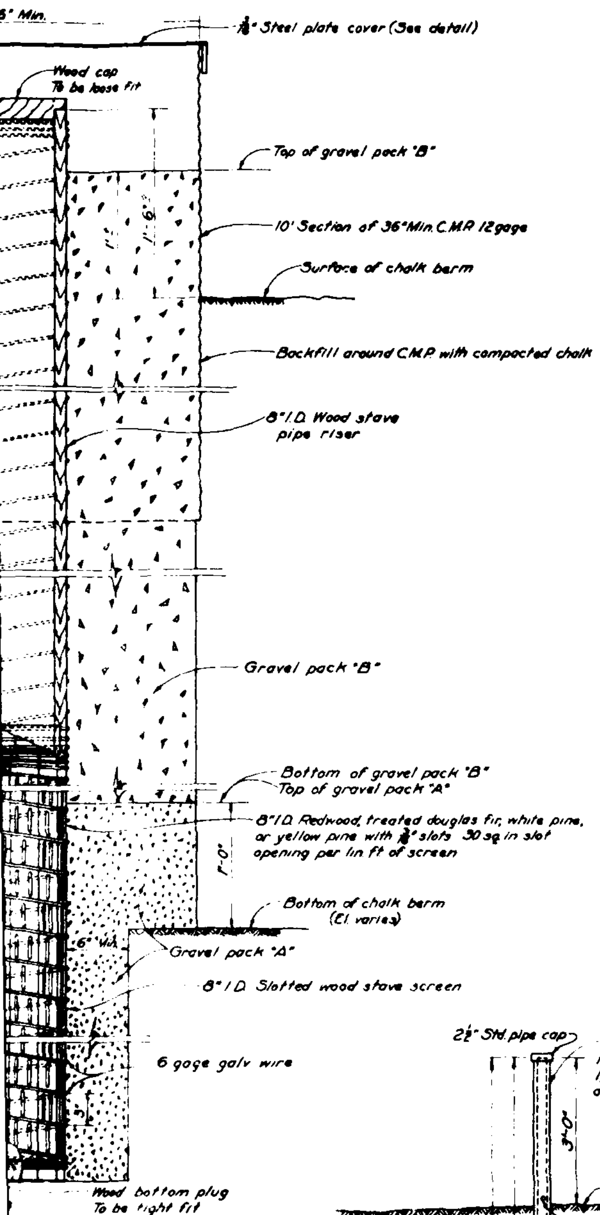
STEEL COVER DETAIL

SCALE 1½ INCHES = 1 FOOT

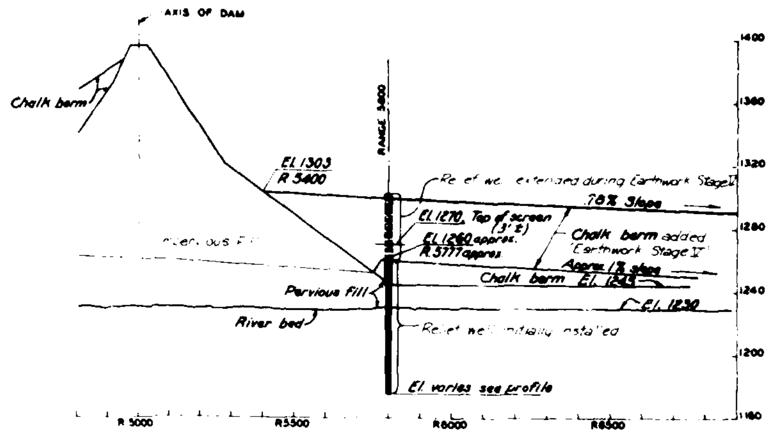


PIEZOMETER T

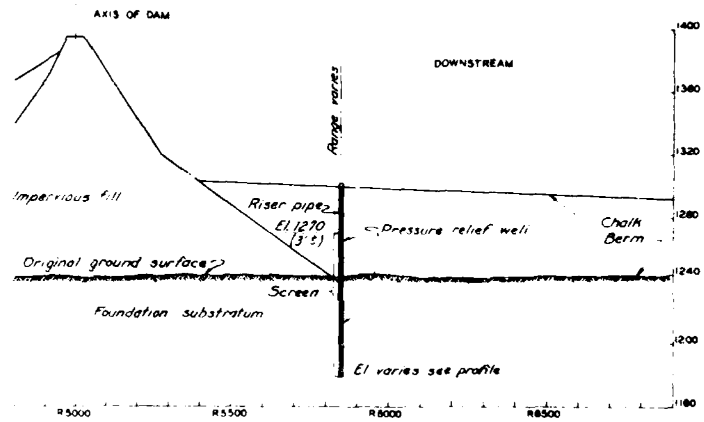
NO SCA



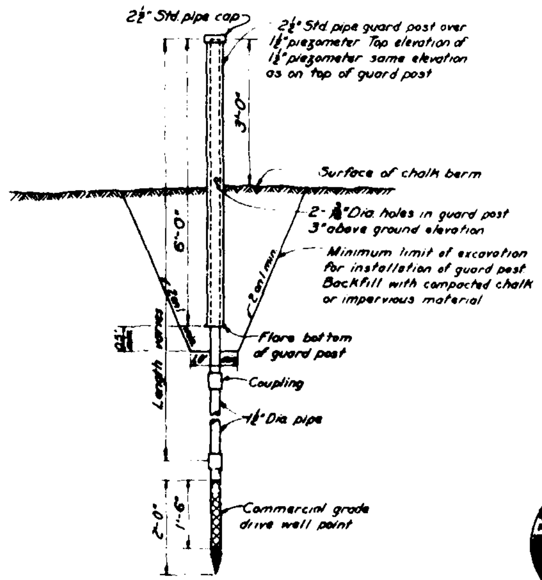
LL DETAILS



TYPICAL SECTION
STA. 60+00 TO STA. 76+00



TYPICAL SECTION
STA. 76+00 TO STA. 93+70
SHOWING LOCATION OF PRESSURE RELIEF WELL



PIEZOMETER TUBE DETAIL
NO SCALE

NOTES:
1. All elevations shown refer to M.S.L. U.S.C. & G.S. 1929 general adjustments

CORPS OF ENGINEERS, U. S. ARMY
OFFICE OF THE DISTRICT ENGINEER
OMAHA DISTRICT
OMAHA, NEBRASKA

MISSOURI RIVER
FORT RANDALL DAM - LAKE FRANCIS CASE
RELIEF WELLS & PIEZOMETER TUBES
DETAILS

DATE	DESCRIPTION	MADE	APP'D

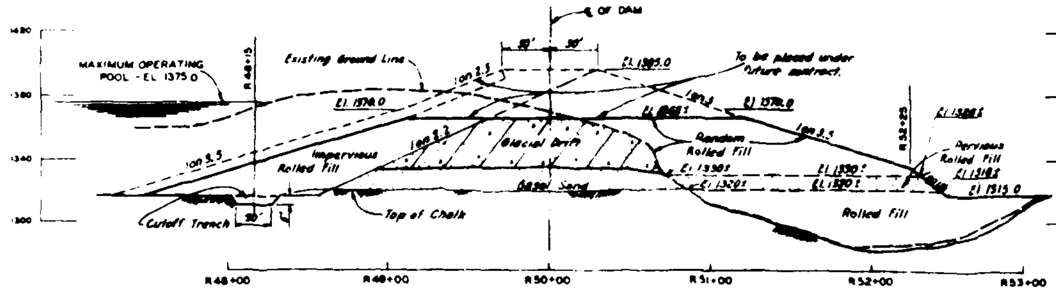
DESIGNED BY:
DRAWN BY:
TRACED BY:
CHECKED BY:
SUBMITTED BY:
APPROVED:
CHIEF, ENGINEERING DIVISION
DATE
SCALE AS SHOWN
SPEC. NO.
DRAWING NUMBER
COL., C. E., DISTRICT ENGINEER

SAFETY PAYS

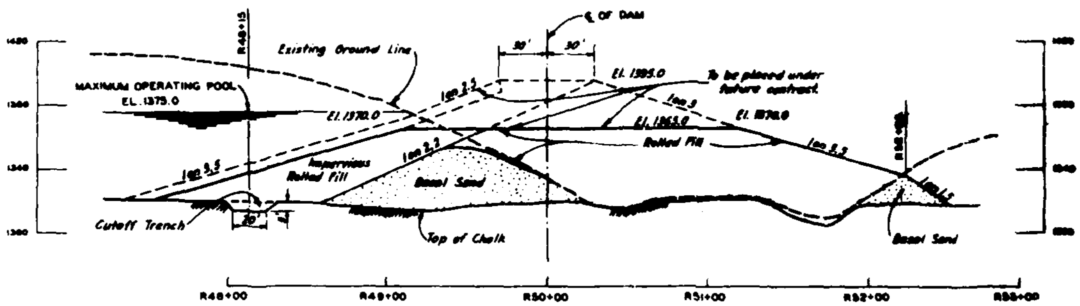
DA-2571

2

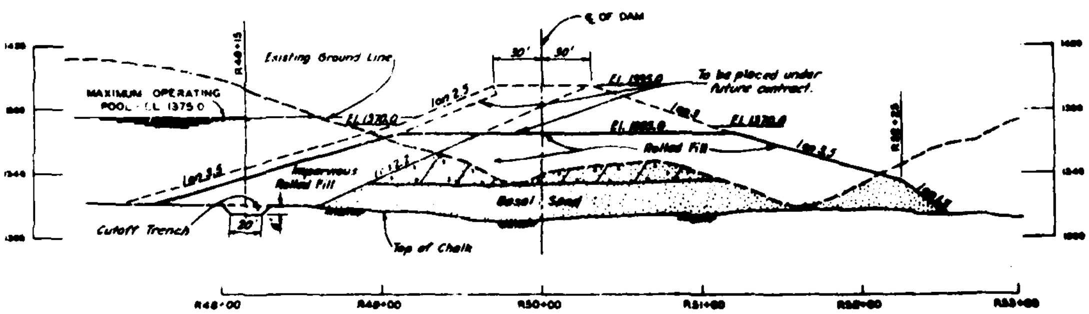
DEPARTMENT OF THE ARMY



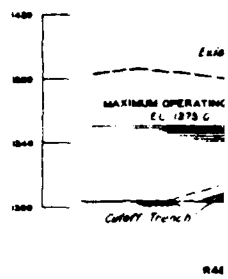
SECTION 1-1
STA. 107+14.85
(ALONG ℄ OF TUNNEL NO 1)

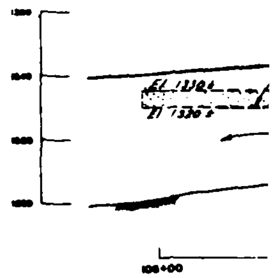


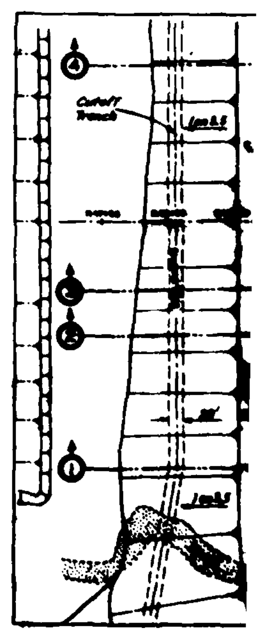
SECTION 2-2
STA. 108+24.85
(ALONG ℄ OF TUNNEL NO. 4)



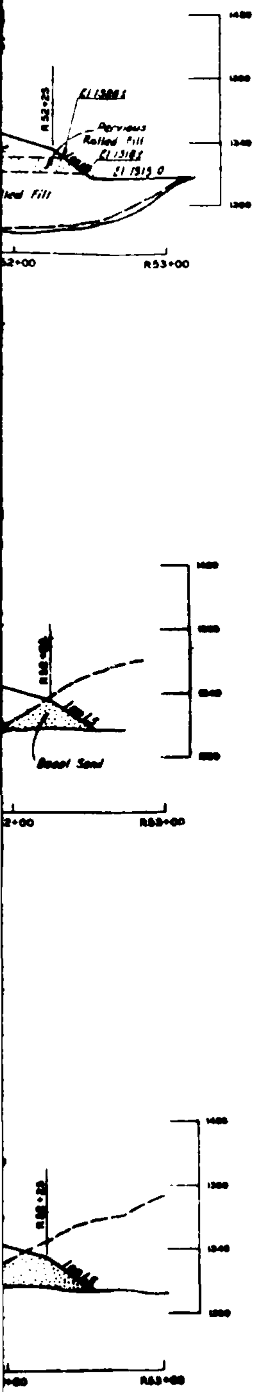
SECTION 3-3
STA. 109+94.85
ALONG ℄ OF TUNNEL NO. 5

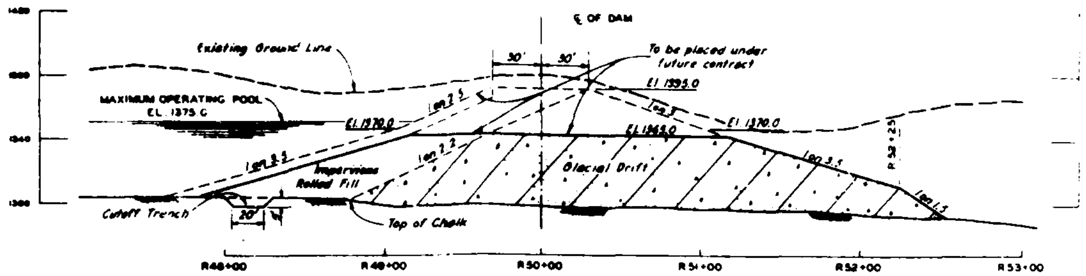






EMB

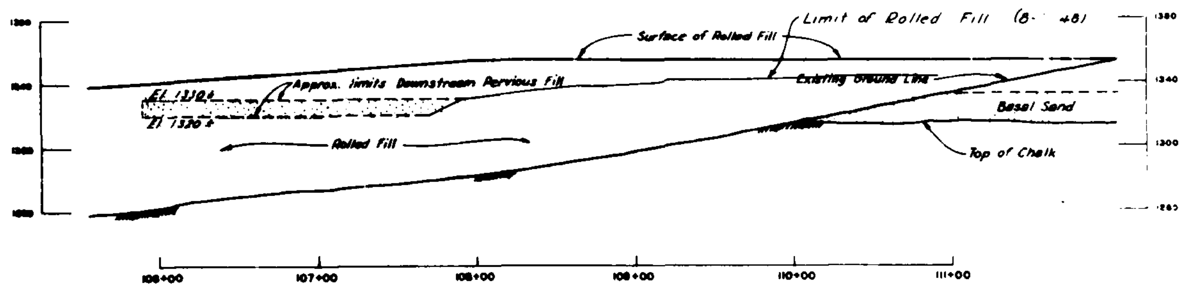




SECTION 4-4

STA. 113+44.86

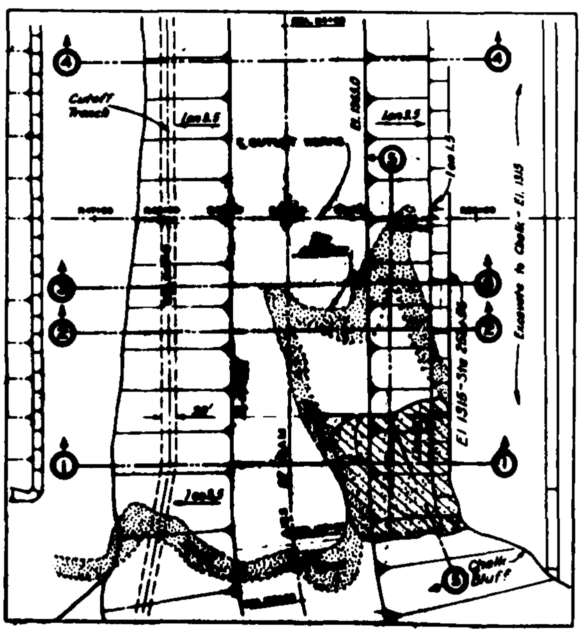
(ALONG ℄ OF TUNNEL NO. 10)



SECTION 5-5

(PROFILE THRU COULEE)

THIS DRAWING HAS BEEN REDUCED TO THREE-EIGHTHS THE ORIGINAL SCALE.



EMBANKMENT PLAN

℄ STA. 1002 TO 1042

SCALE: 1 INCH = 100 FEET

NOTES:

1. For location of plan see Dwg. No. R-156-4D/12.
2. Plan and Section 1-1, and Profile 5-5 show approximate limits of downstream pervious fill. Pervious fill elevations may be adjusted to meet field conditions, but this layer must have a minimum thickness of 10 feet and for it's full height must contact basal sand layer.

THIS PLAN ACCOMPANIES CONTRACT NO.
W-25-066-eng-1434 MODIFICATION NO. 7
THIS DRAWING ADDED TO ORIGINAL FOLIO BY MODIFICATION NO. 7

	DATE	DESCRIPTION	MADE	APPRD
	8-1-50	Revised to show "As-Built" conditions		
M 7	9-11-48	Revised D.S. Pervious Drain		

REVISIONS

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
OFFICE OF THE DISTRICT ENGINEER
OMAHA, NEBRASKA

DRAWN BY: G.T.S.
TRACED BY: R.W.T.
CHECKED BY: B.R.R. / S.T.T.

MISSOURI RIVER
FORT RANDALL RESERVOIR
INITIAL EARTHWORK
DETAILED EMBANKMENT SECTIONS
LEFT ABUTMENT

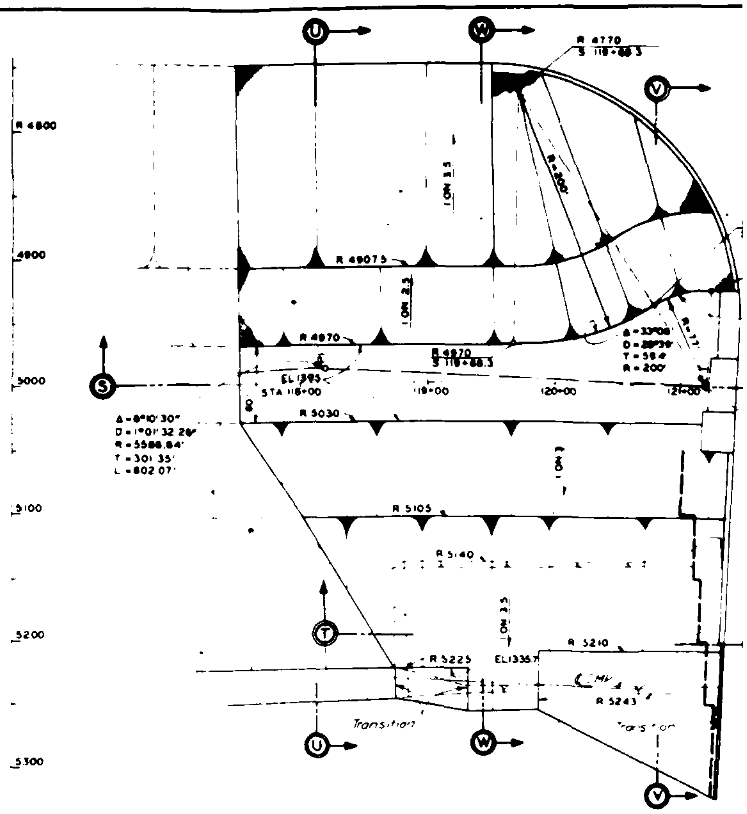
CHIEF, ENGINEERING DIVISION

DATE: JUNE 1946

SCALE AS SHOWN

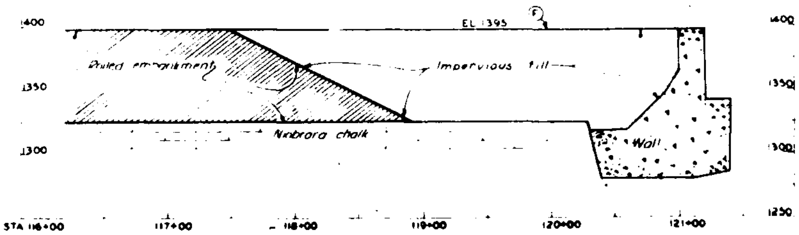
COL., C.E., DISTRICT ENGINEER

DRAWING NUMBER: R156-4/24

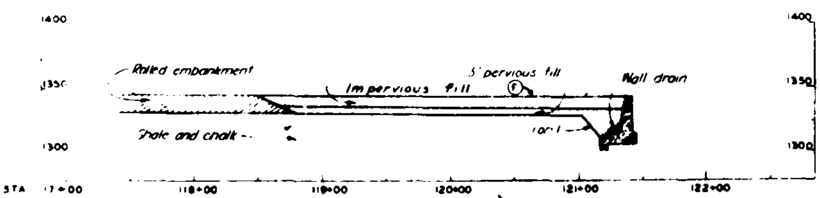


DETAIL - BACKFILL
RIVERWARD END SPILLWAY

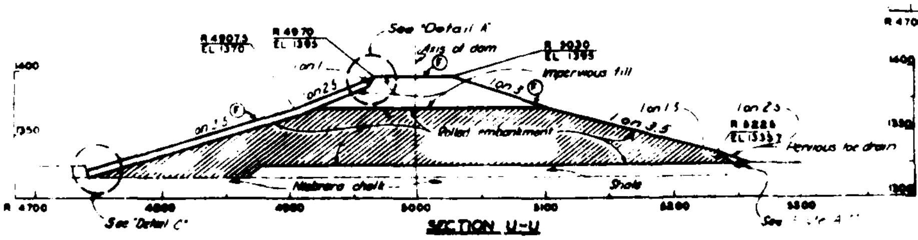
SCALE: 1 INCH = 50 FEET



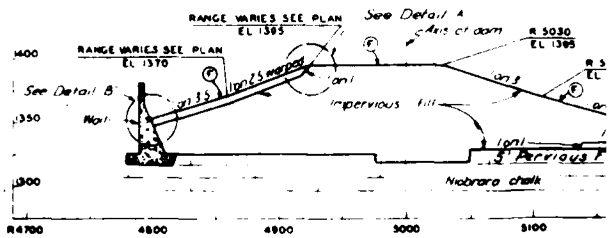
SECTION S-S



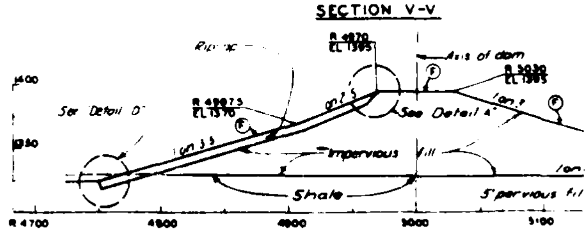
SECTION T-T



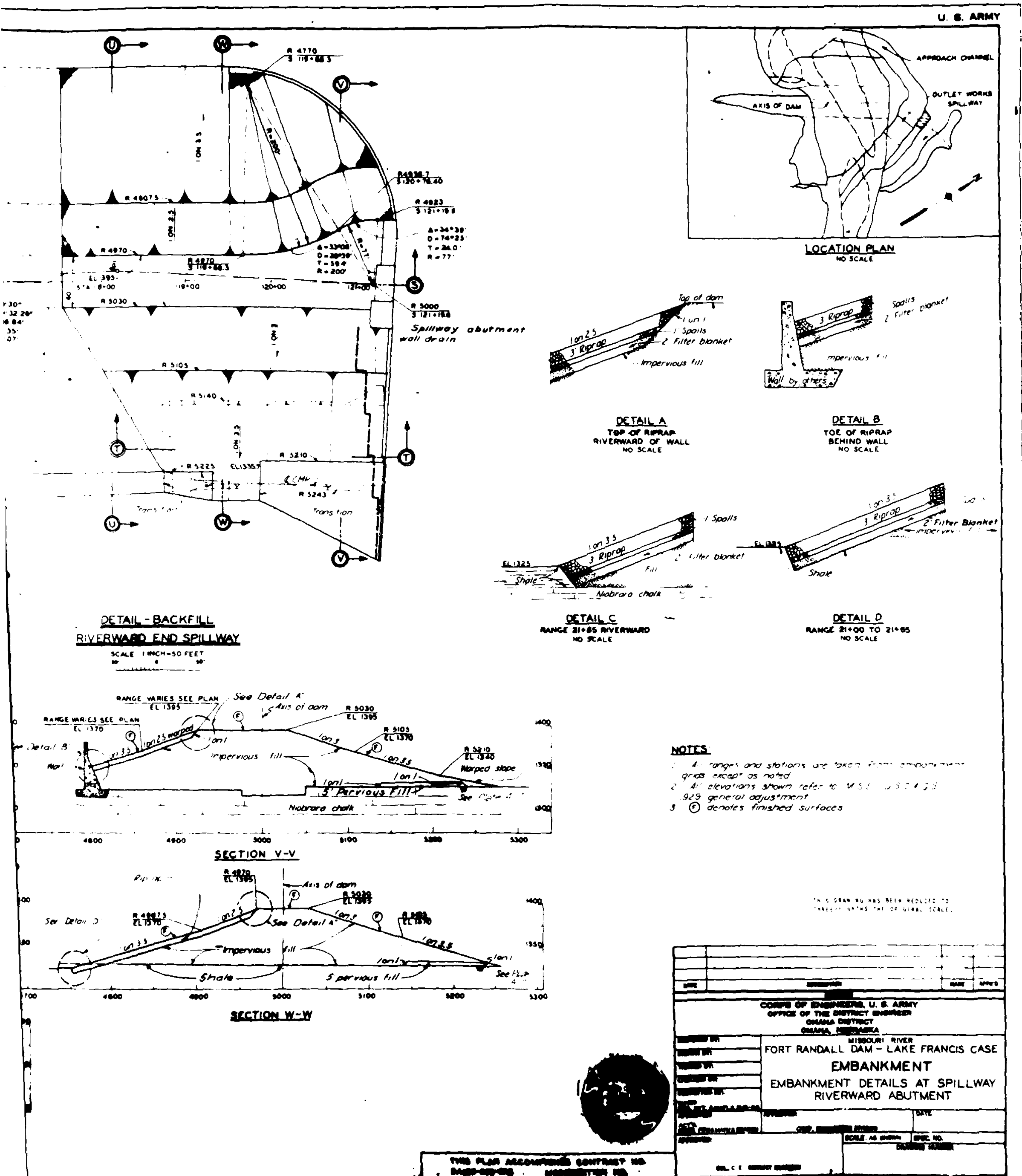
SECTION U-U



SECTION V-V

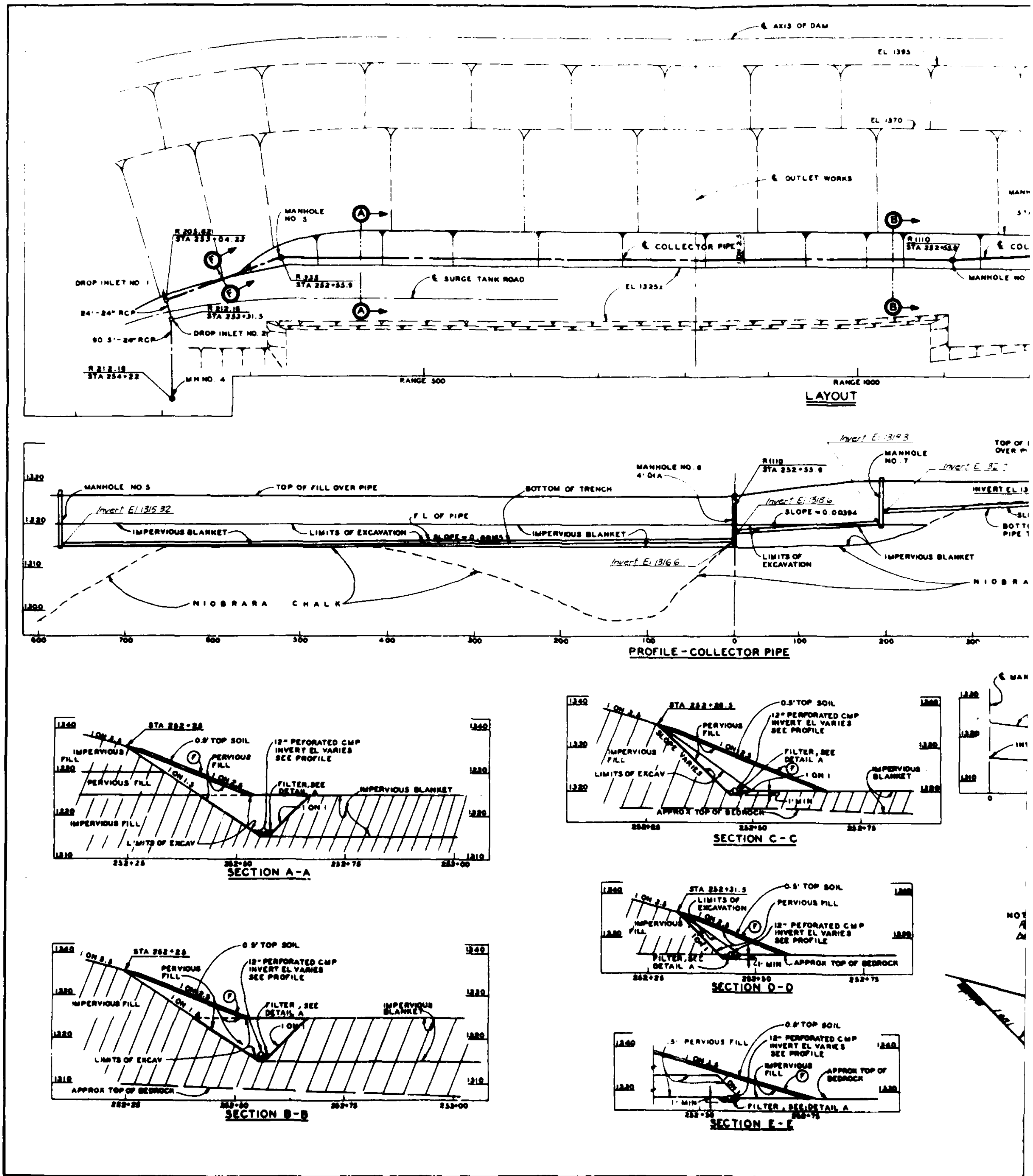


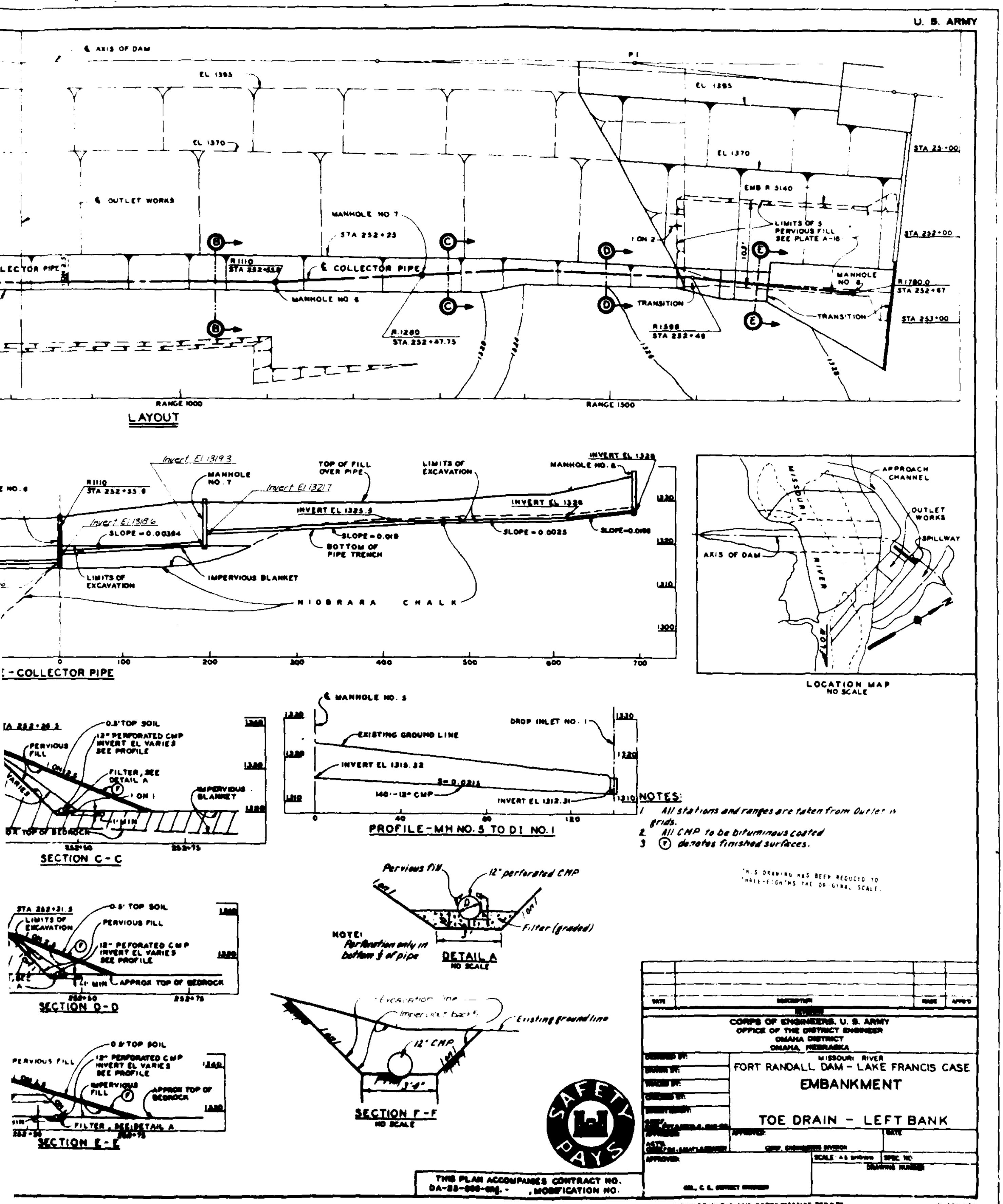
SECTION W-W


U. S. ARMY
APPROACH CHANNEL
AXIS OF DAM
OUTLET WORKS SPILLWAY
LOCATION PLAN
NO SCALE
Spillway abutment wall drain
Top of dam
1 on 2.5
3' Riprap
1' Spalls
2' Filter blanket
Impervious fill
Wall by others
DETAIL A
TOP OF RIPRAP RIVERWARD OF WALL
NO SCALE
DETAIL B
TOE OF RIPRAP BEHIND WALL
NO SCALE
1 on 3.5
EL 1325
Shale
Niobrara chalk
DETAIL C
RANGE 21+85 RIVERWARD
NO SCALE
DETAIL D
RANGE 21+00 TO 21+85
NO SCALE
DETAIL - BACKFILL
RIVERWARD END SPILLWAY
SCALE 1 INCH=50 FEET
RANGE VARIES SEE PLAN
EL 1395
EL 1370
See Detail A
Axis of dam
R 5030
R 5103
R 5210
EL 1340
Warped slope
Impervious fill
5' Pervious Fill
Niobrara chalk
SECTION V-V
SECTION W-W
NOTES:
1. All ranges and stations are taken from embankment grids except as noted.
2. All elevations shown refer to M.S.L. U.S.C.&G.S. 1929 general adjustment.
3. (F) denotes finished surfaces
CORPS OF ENGINEERS, U. S. ARMY
OFFICE OF THE DISTRICT ENGINEER
OMAHA DISTRICT
OMAHA, NEBRASKA
MISSOURI RIVER
FORT RANDALL DAM – LAKE FRANCIS CASE
EMBANKMENT
EMBANKMENT DETAILS AT SPILLWAY
RIVERWARD ABUTMENT
THIS PLAN ACCOMPANIES CONTRACT NO.

2

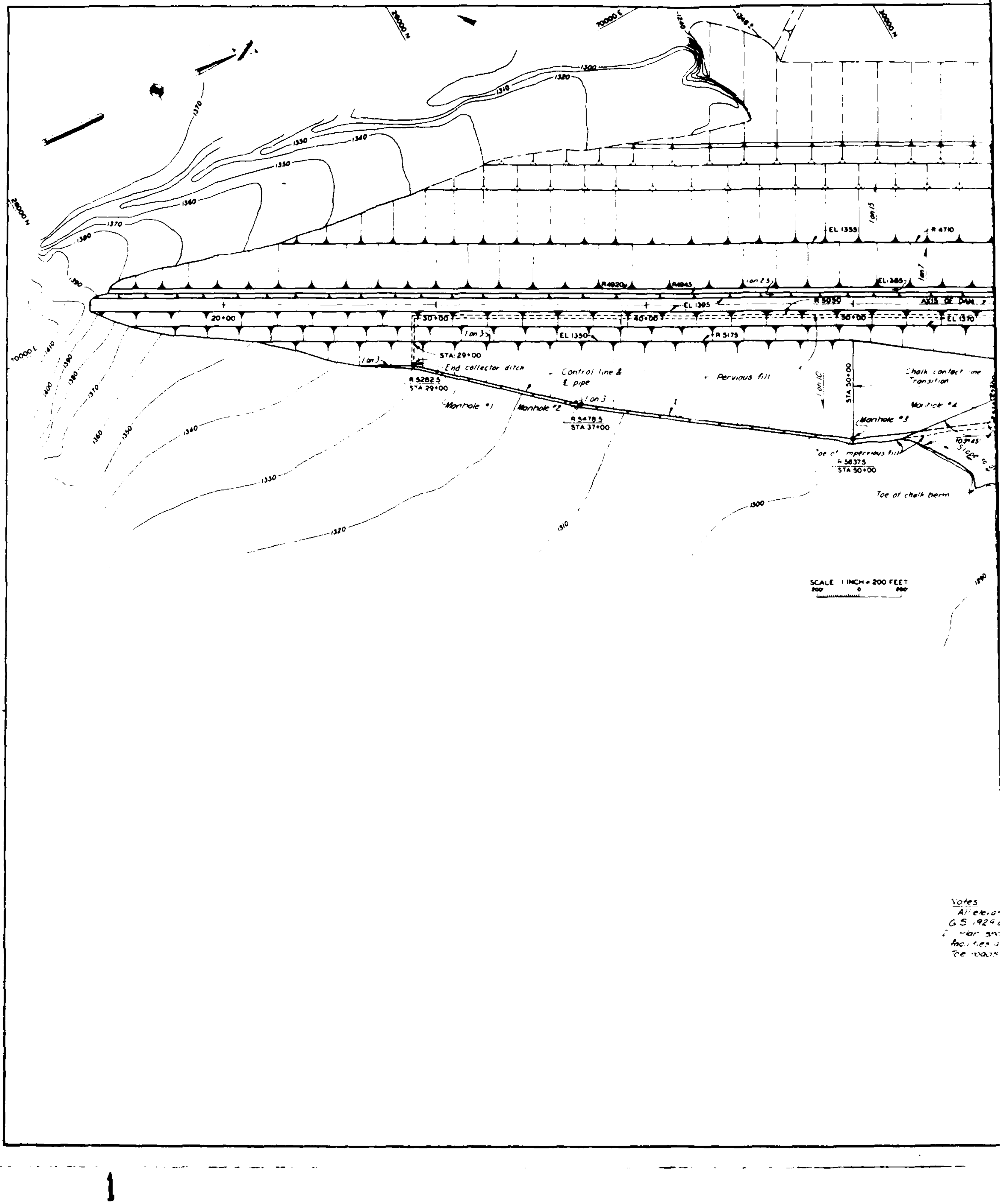
CORPS OF ENGINEERS

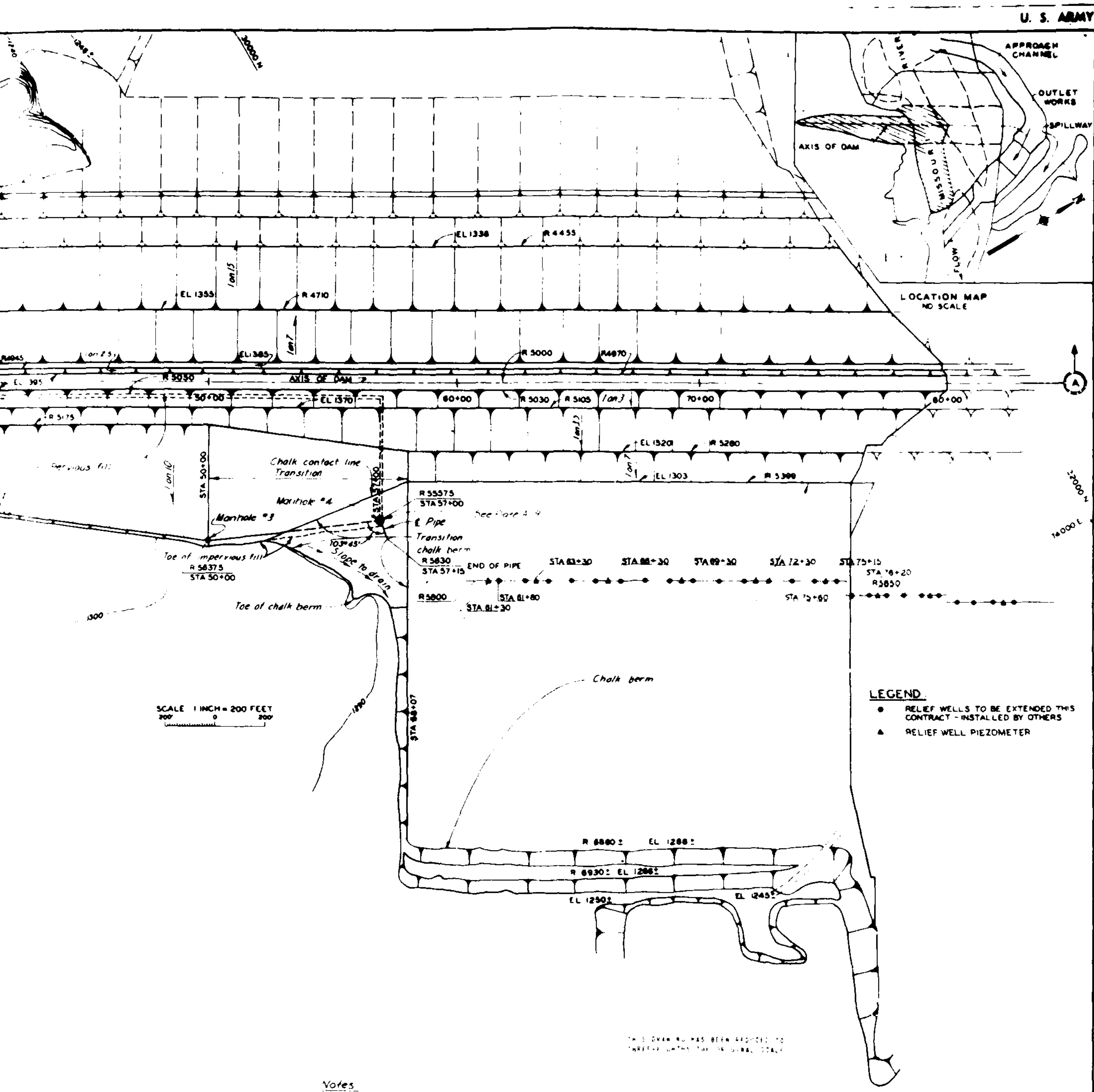



U. S. ARMY
AXIS OF DAM
EL 1395
EL 1370
OUTLET WORKS
MANHOLE NO 7
STA 252+25
COLLECTOR PIPE
MANHOLE NO 6
R 1110 STA 252+55.8
R.1260 STA 252+47.75
R1598 STA 252+49
TRANSITION
MANHOLE NO 8
R1780.0 STA 252+67
LIMITS OF PERVIOUS FILL SEE PLATE A-16
EMB R 5140
STA 25+00
STA 252+00
STA 253+00
RANGE 1000
RANGE 1500
LAYOUT
Invert El 1319.3
Invert El 1321.7
Invert El 1318.6
TOP OF FILL OVER PIPE
LIMITS OF EXCAVATION
INVERT EL 1328
INVERT EL 1325.5
INVERT EL 1326
SLOPE = 0.00384
SLOPE = 0.019
SLOPE = 0.0025
SLOPE = 0.0198
BOTTOM OF PIPE TRENCH
IMPERVIOUS BLANKET
NIOBRARA CHALK
COLLECTOR PIPE
APPROACH CHANNEL
OUTLET WORKS
SPILLWAY
AXIS OF DAM
MISSOURI RIVER
LOCATION MAP
NO SCALE
0.5' TOP SOIL
12" PERFORATED CMP INVERT EL VARIES SEE PROFILE
PERVIOUS FILL
FILTER, SEE DETAIL A
IMPERVIOUS BLANKET
SECTION C-C
MANHOLE NO. 5
EXISTING GROUND LINE
INVERT EL 1315.32
DROP INLET NO. 1
140'-12" CMP
INVERT EL 1312.31
PROFILE-MH NO. 5 TO DI NO. 1
NOTES:
1 All stations and ranges are taken from Outlet grids.
2 All CMP to be bituminous coated
3 (F) denotes finished surfaces.
THIS DRAWING HAS BEEN REDUCED TO THREE-EIGHTHS THE ORIGINAL SCALE.
Pervious fill
12" perforated CMP
Filter (graded)
NOTE: Perforation only in bottom ⅓ of pipe
DETAIL A
NO SCALE
LIMITS OF EXCAVATION
12" PERFORATED CMP INVERT EL VARIES SEE PROFILE
APPROX TOP OF BEDROCK
SECTION D-D
Excavation line
Impervious backfill
Existing ground line
12" CMP
SECTION F-F
NO SCALE
IMPERVIOUS FILL
FILTER, SEE DETAIL A
SECTION E-E
SAFETY PAYS
CORPS OF ENGINEERS, U. S. ARMY
OFFICE OF THE DISTRICT ENGINEER
OMAHA DISTRICT
OMAHA, NEBRASKA
MISSOURI RIVER
FORT RANDALL DAM - LAKE FRANCIS CASE
EMBANKMENT
TOE DRAIN - LEFT BANK
THIS PLAN ACCOMPANIES CONTRACT NO. DA-25-066-eng- , MODIFICATION NO.

2

CORPS OF ENGINEERS





Notes
1. All elevations shown refer to MSL, USC and GS 1929 general adjustment
2. Plan shows right bank embankment and drain facilities at completion of embankment construction. Toe roads are not shown

CORPS OF ENGINEERS, U. S. ARMY
OFFICE OF THE DISTRICT ENGINEER
OMAHA DISTRICT
OMAHA, NEBRASKA

MISSOURI RIVER
FORT RANDALL DAM – LAKE FRANCIS CASE
EMBANKMENT
PLAN – RIGHT BANK

SAFETY PAYS

THIS PLAN ACCOMPANIES CONTRACT NO.
DA-25-066-eng. , MODIFICATION NO.

CORPS OF ENGINEERS

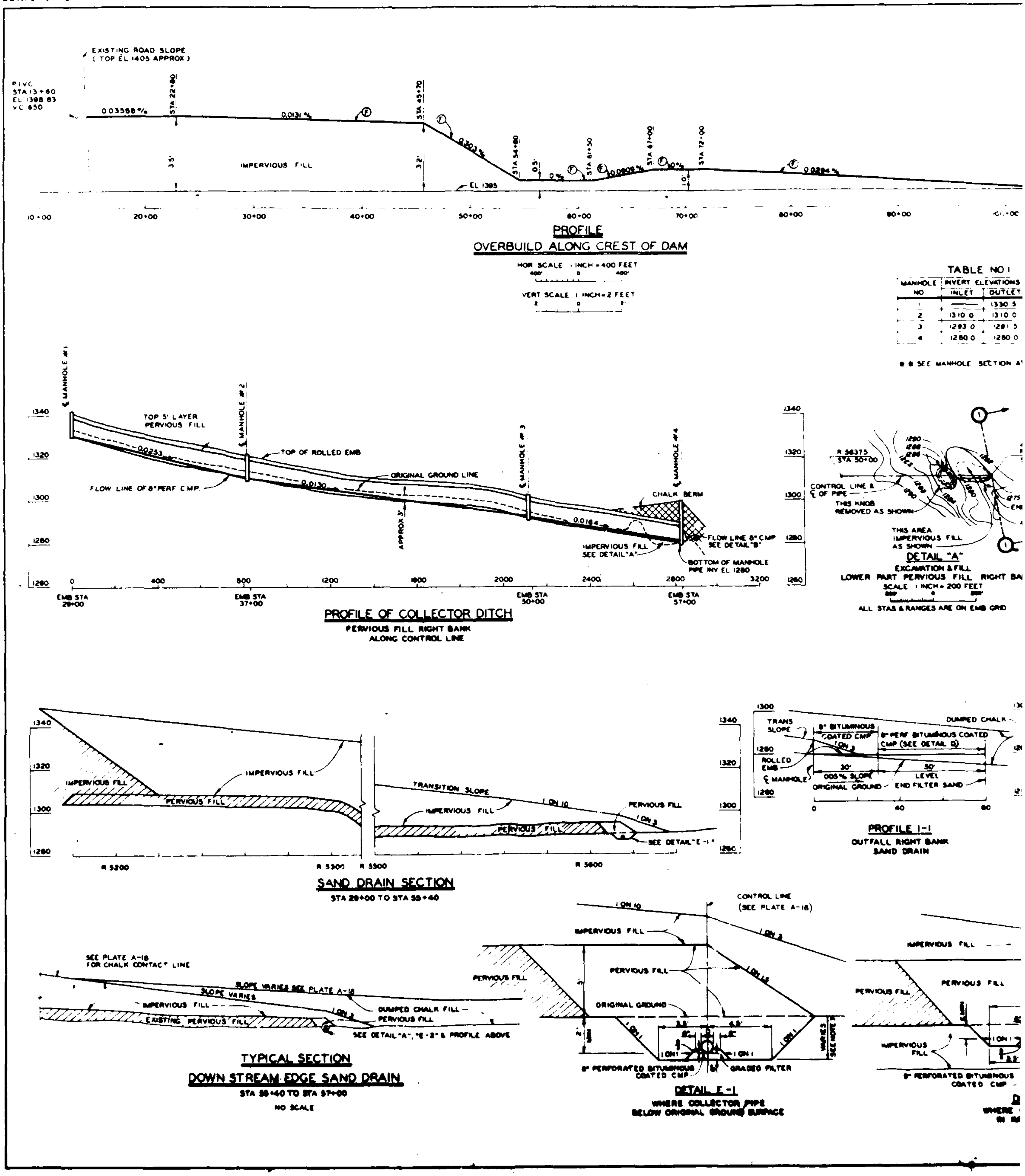


TABLE NO 1

MANHOLE NO	INVERT ELEVATIONS INLET	OUTLET
1	—	1330.5
2	1310.0	1310.0
3	1293.0	1291.5
4	1280.0	1280.0

* SEE MANHOLE SECTION A

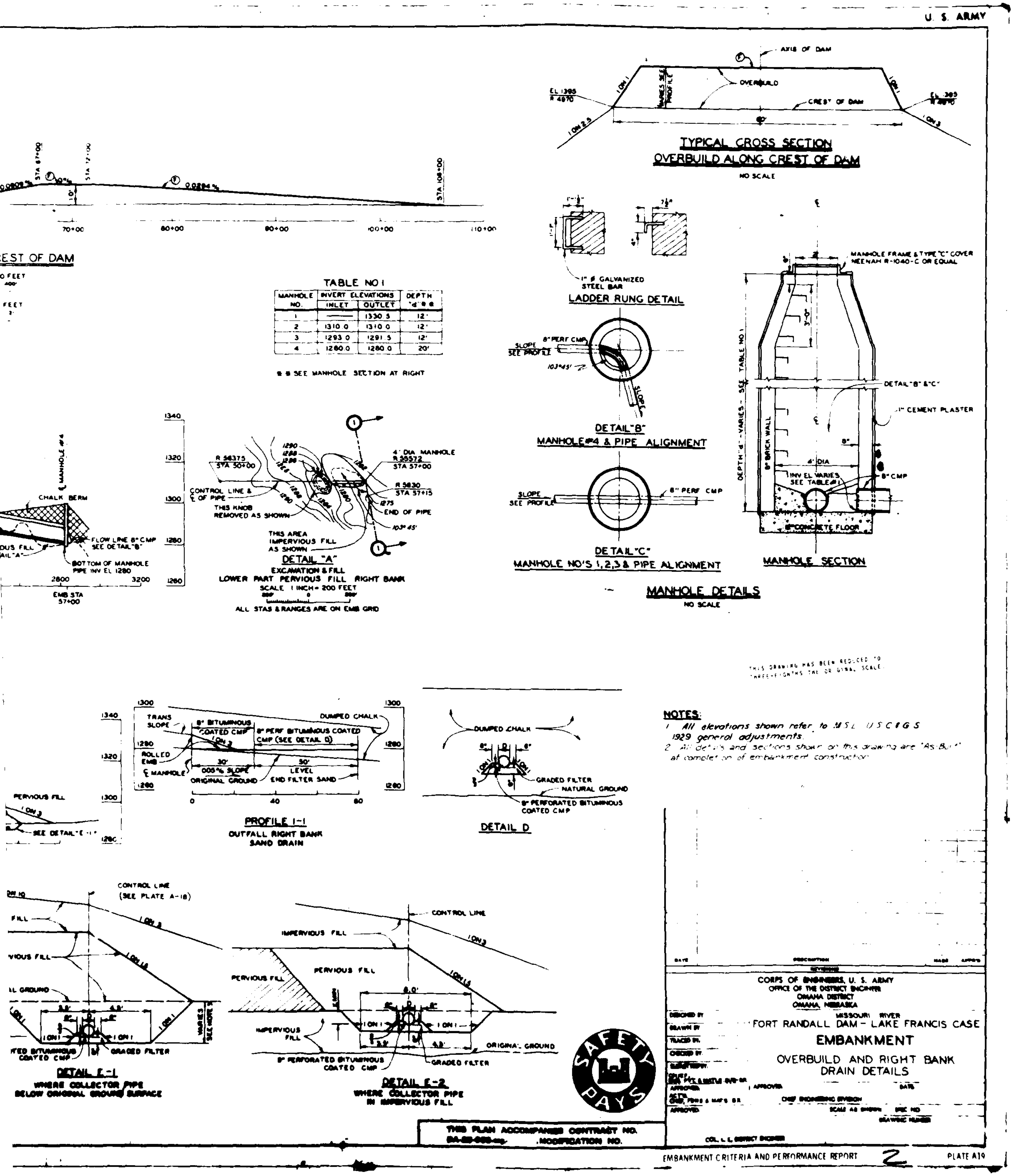


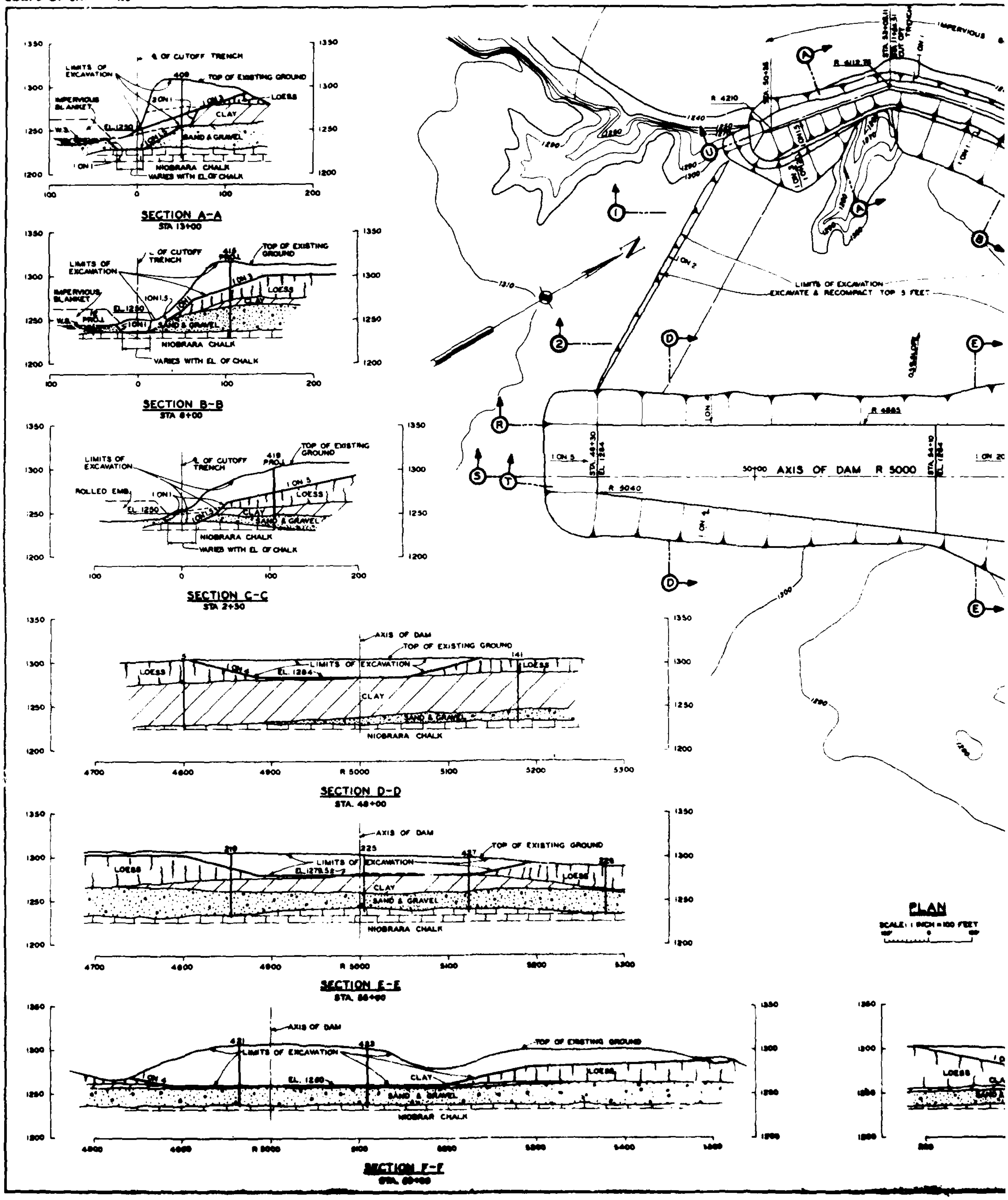
TABLE NO I

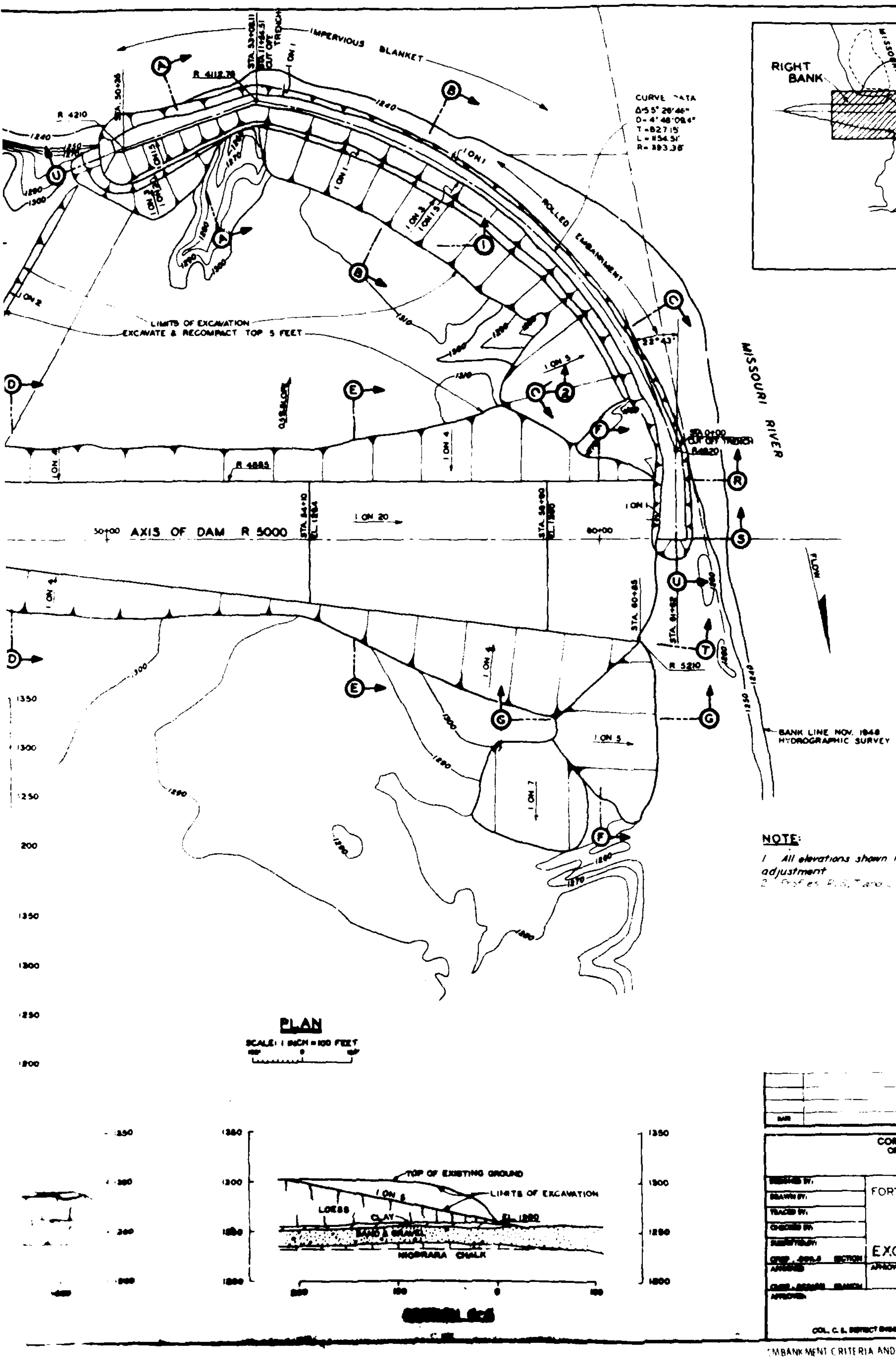
MANHOLE NO.	INVERT ELEVATIONS INLET	OUTLET	DEPTH "d" * *
1	——	1330.5	12'
2	1310.0	1310.0	12'
3	1293.0	1291.5	12'
4	1280.0	1280.0	20'

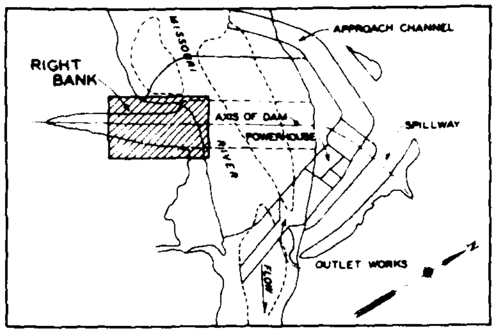
* * SEE MANHOLE SECTION AT RIGHT

NOTES:
1. All elevations shown refer to M.S.L. U.S.C.&G.S. 1929 general adjustments.
2. All details and sections shown on this drawing are "As Built" at completion of embankment construction.

CORPS OF ENGINEERS







LOCATION MAP
NO SCALE

NOTE:

1. All elevations shown refer to M.S.L.-U.S.C.&G.S. 1929 general adjustment

CORPS OF ENGINEERS, U. S. ARMY
OFFICE OF THE DISTRICT ENGINEER
OMAHA DISTRICT
OMAHA, NEBRASKA

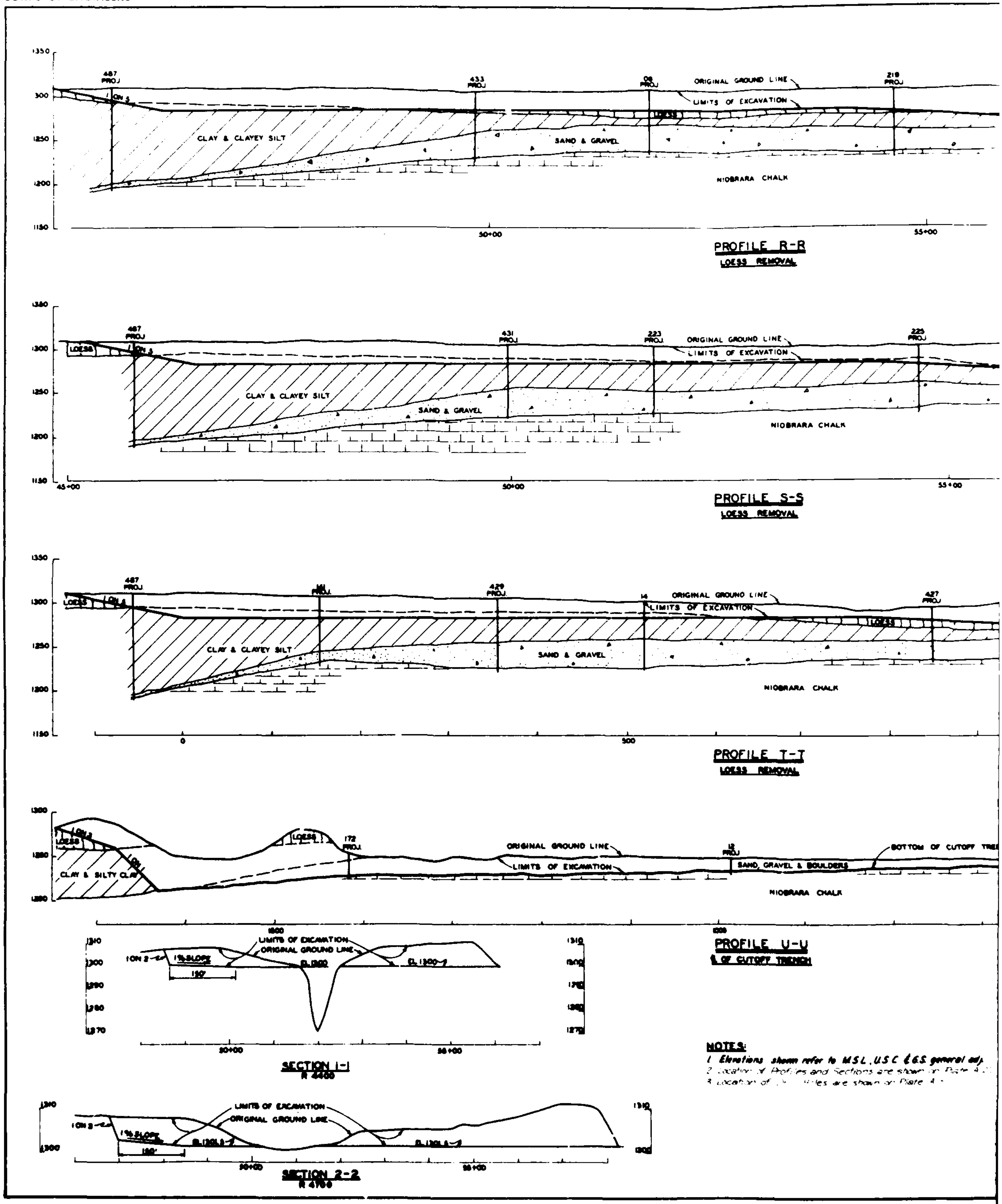
MISSOURI RIVER
FORT RANDALL DAM - LAKE FRANCIS CASE
EMBANKMENT
RIGHT BANK
EXCAVATION PLAN AND SECTIONS

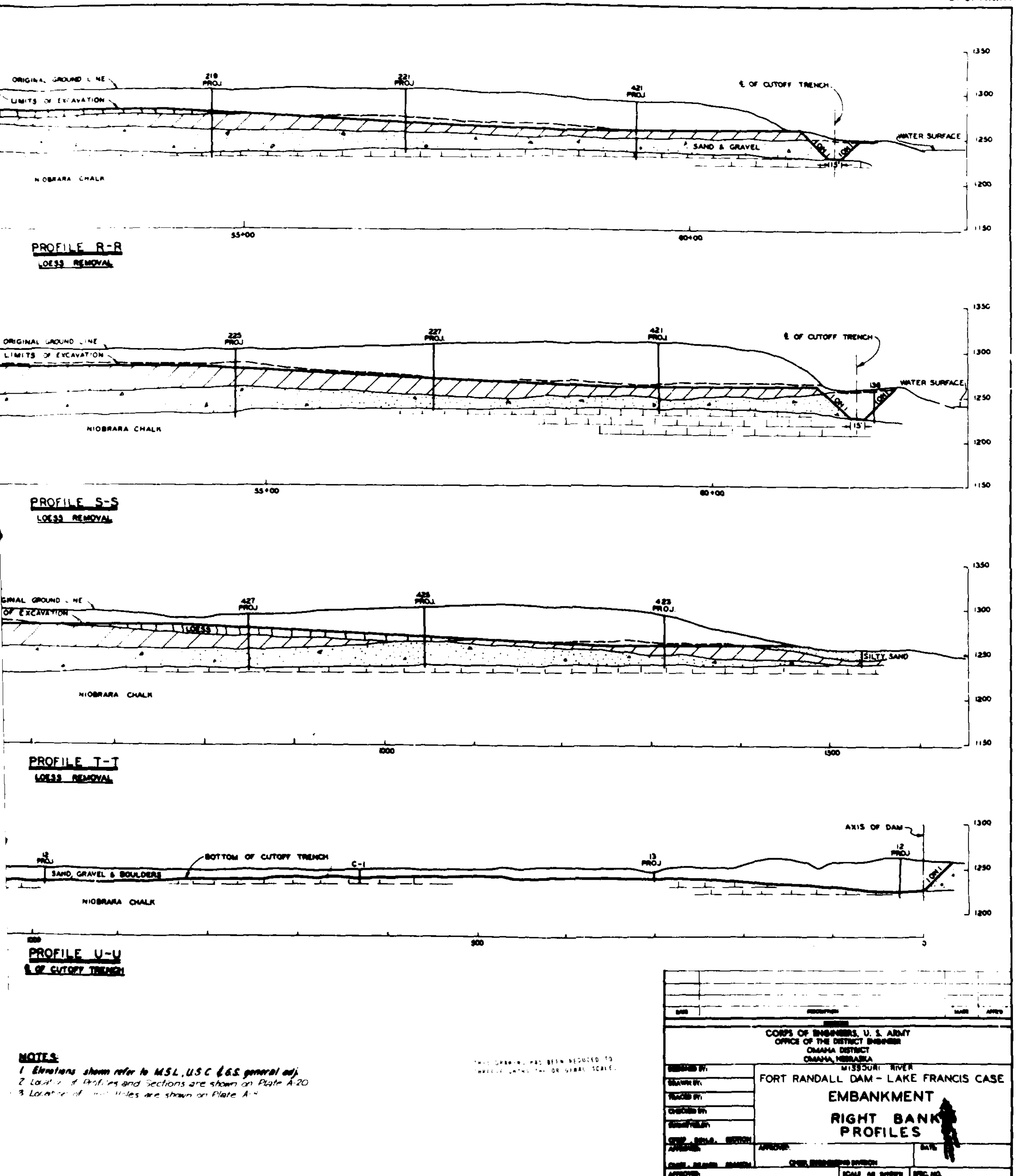
SCALE: AS SHOWN

COL. C. E. DISTRICT ENGINEER

2

CORPS OF ENGINEERS





PROFILE R-R
LOESS REMOVAL

PROFILE S-S
LOESS REMOVAL

PROFILE T-T
LOESS REMOVAL

PROFILE U-U
℄ OF CUTOFF TRENCH

NOTES:

1. *Elevations shown refer to M.S.L., U.S.C. & G.S. general adj.*
2. *Location of Profiles and Sections are shown on Plate A20*
3. *Location of [illegible] Holes are shown on Plate A[illegible]*

THIS DRAWING HAS BEEN REDUCED TO [illegible]

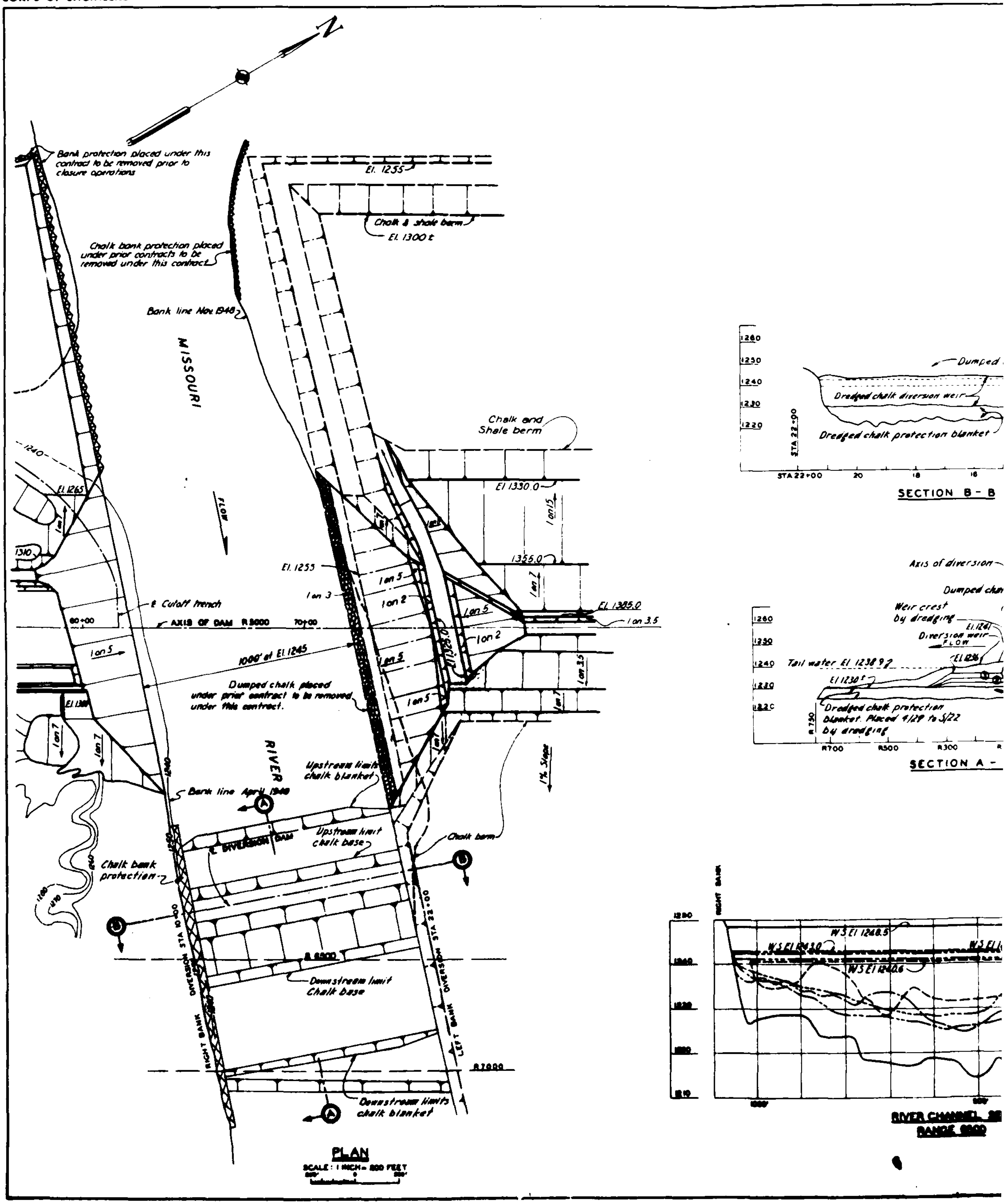
CORPS OF ENGINEERS, U. S. ARMY
OFFICE OF THE DISTRICT ENGINEER
OMAHA DISTRICT
OMAHA, NEBRASKA

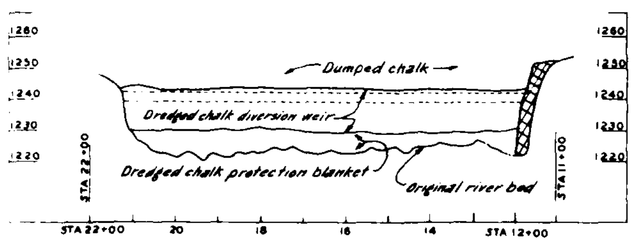
MISSOURI RIVER
FORT RANDALL DAM - LAKE FRANCIS CASE
EMBANKMENT
RIGHT BANK
PROFILES

SCALE AS SHOWN

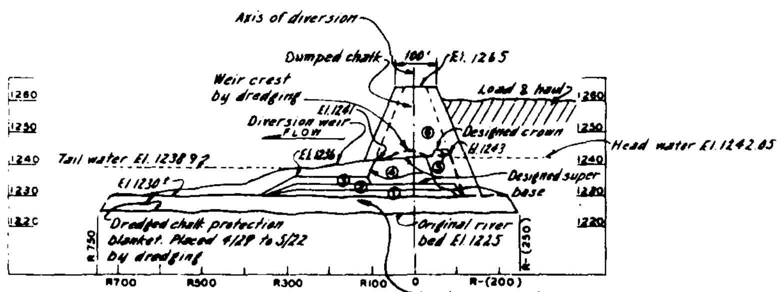
2

CORPS OF ENGINEERS





SECTION B - B

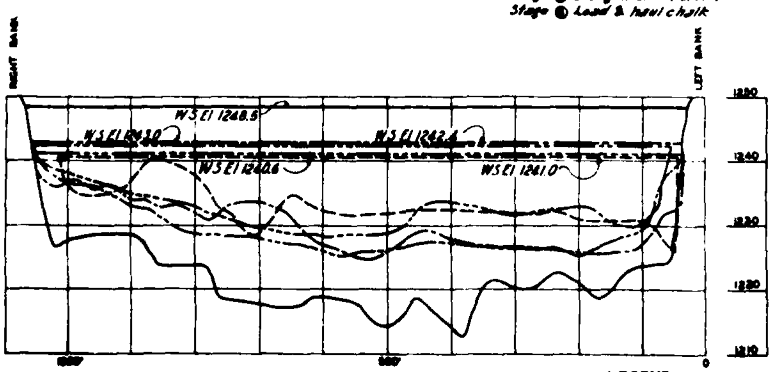


SECTION A - A

NOTE:

DESIGNED STAGES - DIVERSION WEIR

- Stage ① Dredged chalk base
- Stage ② Dredged chalk base
- Stage ③ Dredged chalk base
- Stage ④ Dredged chalk super base
- Stage ⑤ Dredged chalk crown
- Stage ⑥ Load & haul chalk



RIVER CHANNEL SECTION
RANGE 8800

LEGEND

- ———— APRIL, 1949
- — — — MAY, 1949
- — - — JUNE, 1949
- — — - — JULY, 1949
- ——— - - - AUG & SEPT, 1949

REFERENCE DRAWINGS:

NOTES

1. All elevations shown refer to M.S.L. U.S.C.&G.S. general adjustment
2. Side slopes of diversion dam shall be at natural angle of repose as determined in the field

THIS DRAWING HAS BEEN REDUCED TO ... THE ORIGINAL SCALE

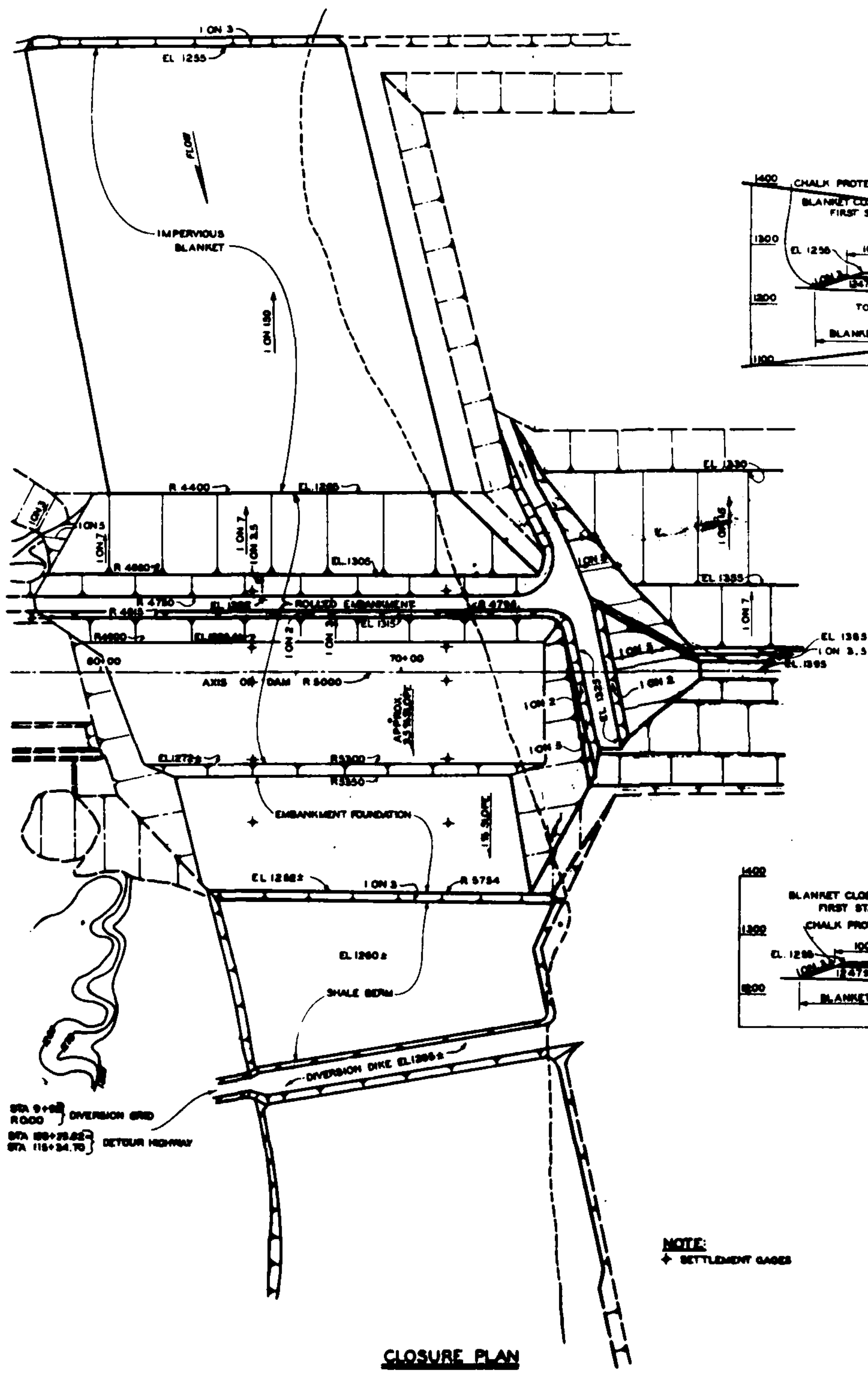
2-2-56	Revised to show "As Built" conditions		
DATE	DESCRIPTION	MADE	APPD

CORPS OF ENGINEERS, U. S. ARMY
OFFICE OF THE DISTRICT ENGINEER
OMAHA DISTRICT
OMAHA, NEBRASKA

MISSOURI RIVER
FORT RANDALL RESERVOIR
EARTHWORK
STAGE III
DIVERSION
PLAN & SECTIONS

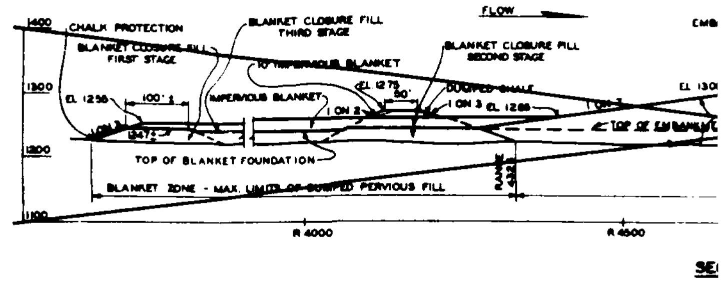
DESIGNED BY: J.R.G.
DRAWN BY: H. W. B.
TRACED BY: H.J.M.
CHECKED BY: J.R.G.
SUBMITTED BY:
RECOMMENDED:
APPROVED:
APPROVED: Henry J. Hoeffer
COL., C. E., DISTRICT ENGINEER
DATE: MARCH 1950
SCALE: AS SHOWN
SPEC. NO.
DRAWING NUMBER: MRI4-31E15.1
SHEET 16 OF 35

2



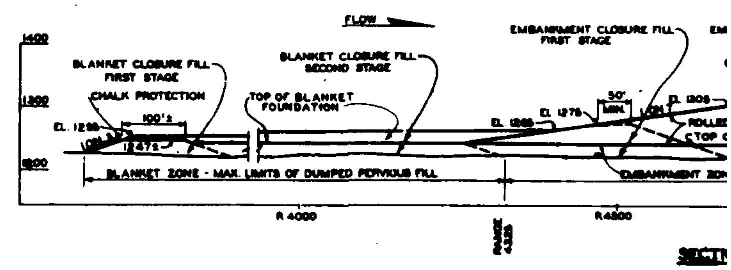
CLOSURE PLAN
AT END OF CLOSURE CONSTRUCTION YEAR

SCALE: 1 INCH = 200 FEET



~~**ORDER OF CONSTRUCTION:**~~

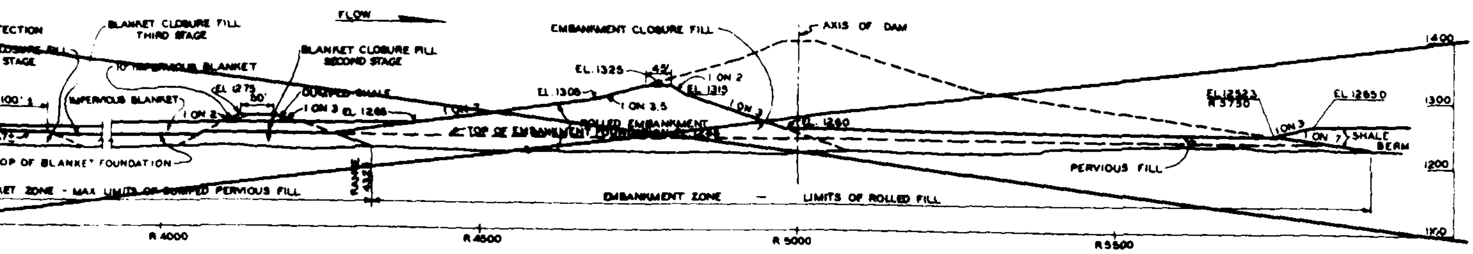
~~1. Blanket Closure Fill, First Stage. To permit lowering of the river level in the blanket and embankment areas to tailwater level.~~
~~2. Blanket Closure Fill, Second Stage. To provide flood protection for the rolled embankment construction and permit installation of unwatering equipment.~~
~~3. Embankment Closure Fill upstream of Range 5000.~~
~~4. Balance of Blanket Closure Fill, and Embankment Closure Fill downstream of Range 5000.~~



ORDER OF CONSTRUCTION:

1. Blanket Closure Fill, First Stage. To permit lowering of the river level in the blanket and embankment areas to tailwater level.
2. Embankment Closure Fill, First Stage. To provide flood protection for the rolled embankment construction.
3. Balance of Embankment Closure Fill upstream of Range 5000.
4. Balance of Blanket Closure Fill, and Embankment Closure Fill downstream of Range 5000.

1



SECTION THRU CLOSURE

EMBANKMENT FOUNDATION FILL PLACED IN UNWATERED RIVER CHANNEL

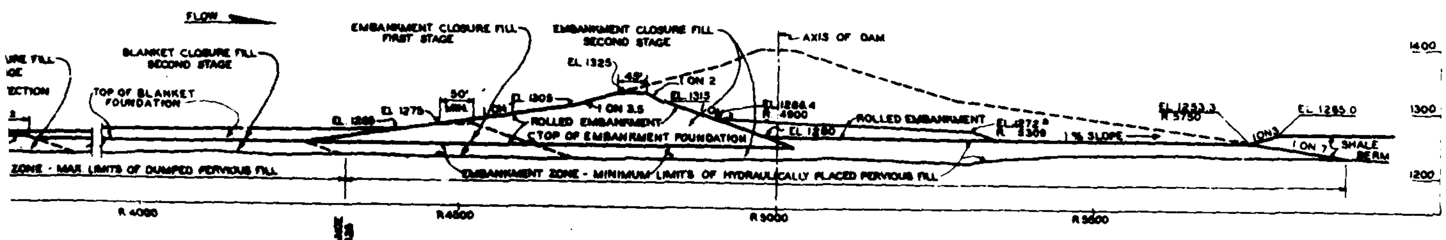
SCALE 1 INCH = 100 FEET

~~ORDER OF CONSTRUCTION:~~

~~1. Blanket Closure Fill, First Stage. To permit lowering of the river level in the blanket and embankment areas to tailwater level.~~
~~2. Blanket Closure Fill, Second Stage. To provide flood protection for the rolled embankment construction and permit installation of unwatering equipment.~~
~~3. Embankment Closure Fill upstream of Range 5000.~~
~~4. Balance of Blanket Closure Fill, and Embankment Closure Fill downstream of Range 5000.~~

NOTES:

1. Simultaneous construction of the various stages will be permitted to facilitate full use of construction equipment.
2. Pervious fill shall be placed to the minimum height above water which will permit satisfactory construction of the impervious blanket and rolled embankment.
3. All elevations shown refer to M.S.L., U.S.C.& G.S. general adj.



SECTION THRU CLOSURE

EMBANKMENT FOUNDATION FILL PLACED IN WATER

SCALE: 1 INCH = 100 FEET

THIS DRAWING HAS BEEN REDUCED TO THREE-EIGHTHS THE ORIGINAL SCALE.

ORDER OF CONSTRUCTION:

1. Blanket Closure Fill, First Stage. To permit lowering of the river level in the blanket and embankment areas to tailwater level.
2. Embankment Closure Fill, First Stage. To provide flood protection for the rolled embankment construction.
3. Balance of Embankment Closure Fill upstream of Range 5000.
4. Balance of Blanket Closure Fill, and Embankment Closure Fill downstream of Range 5000.

2-2-55	Revised to show "As Built" conditions	
10-11-52	Revised encircled items	

CORPS OF ENGINEERS, U. S. ARMY
OFFICE OF THE DISTRICT ENGINEER
OMAHA DISTRICT
OMAHA, NEBRASKA

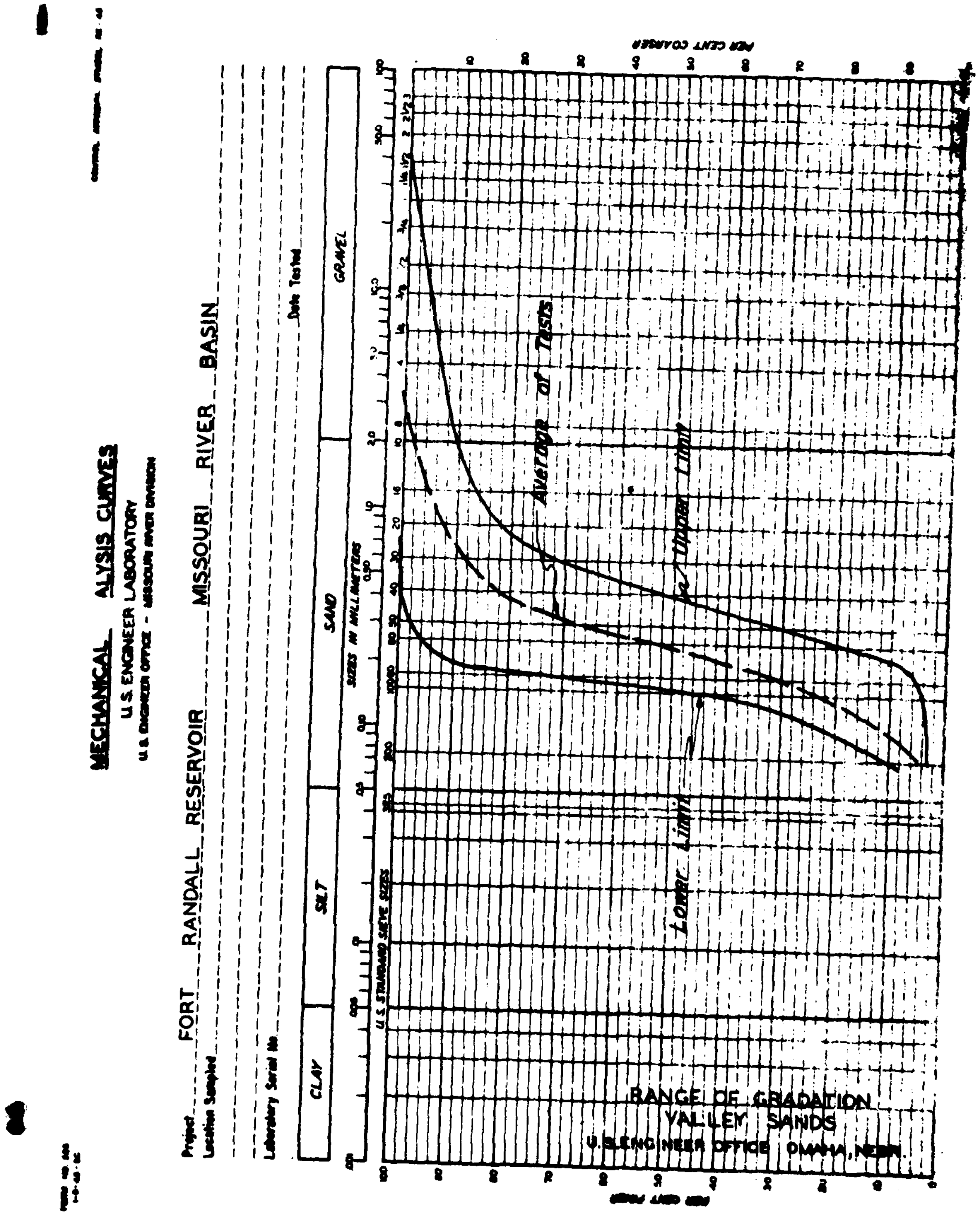
MISSOURI RIVER
FORT RANDALL RESERVOIR
EARTHWORK
STAGE III
CLOSURE
PLAN & SECTIONS

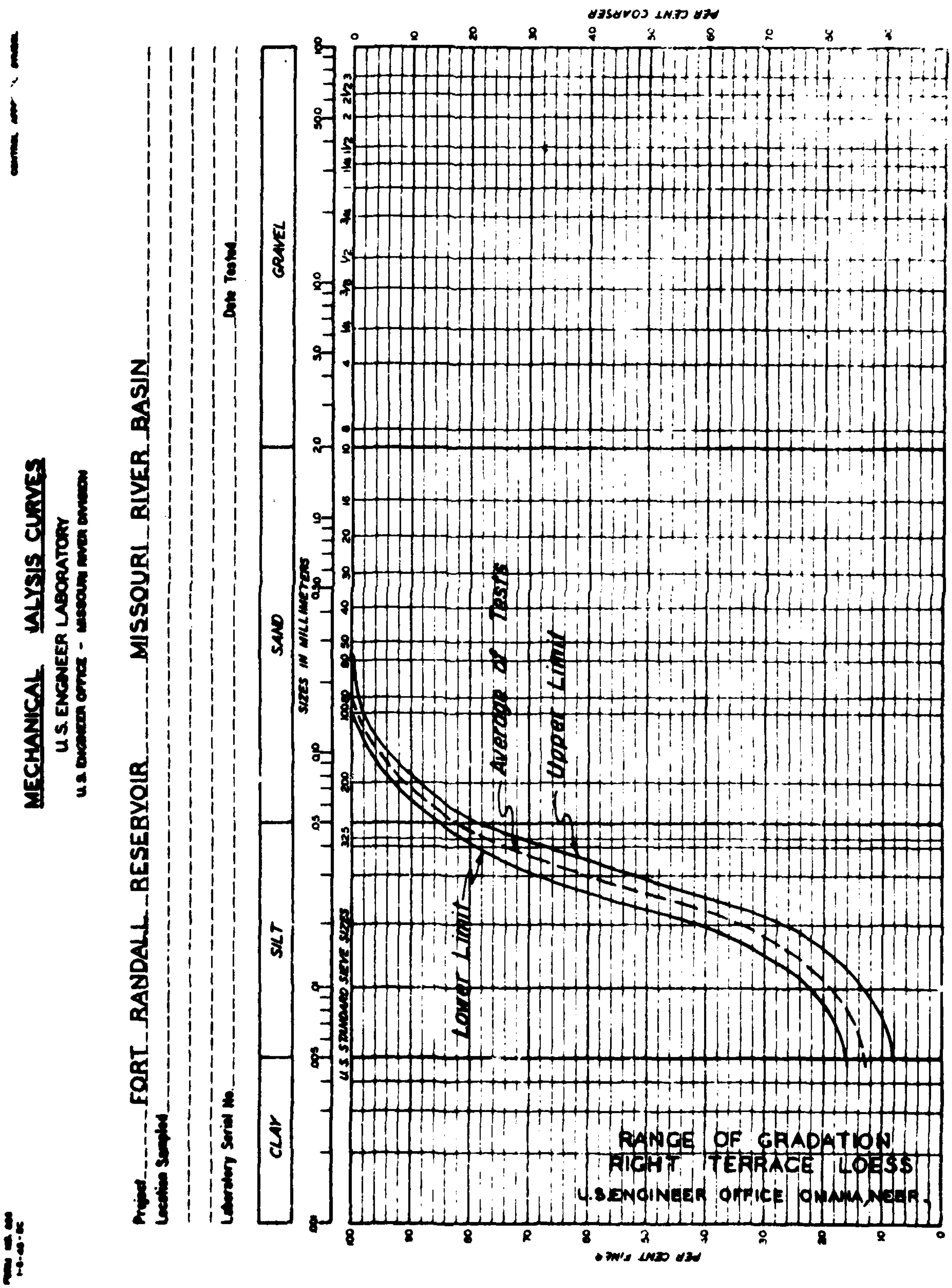
MARCH 1950

MR14-31E18.2

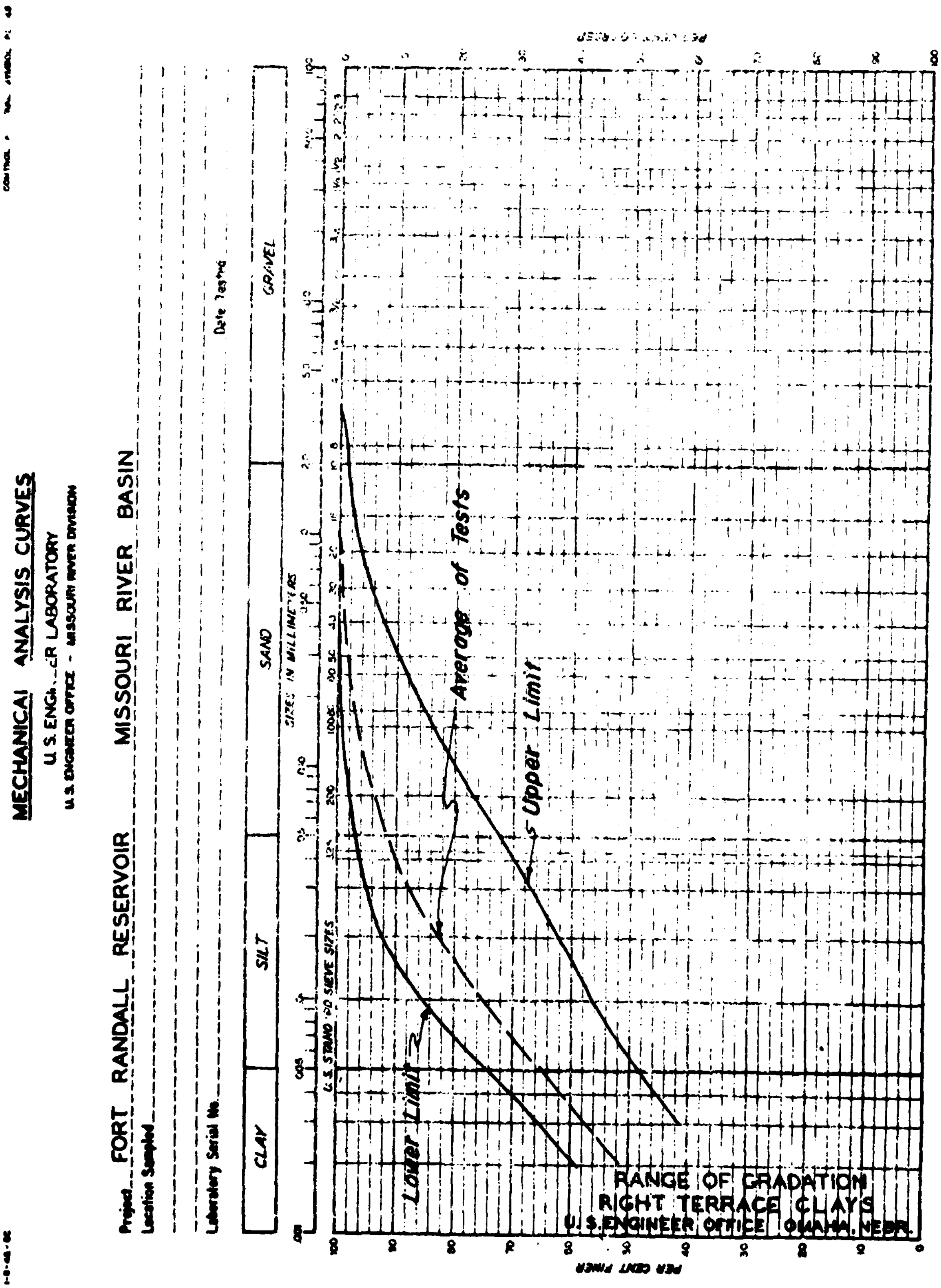
THIS PLAN ACCOMPANIES CONTRACT NO. DA-25-066-ENG-369, MODIFICATION NO.

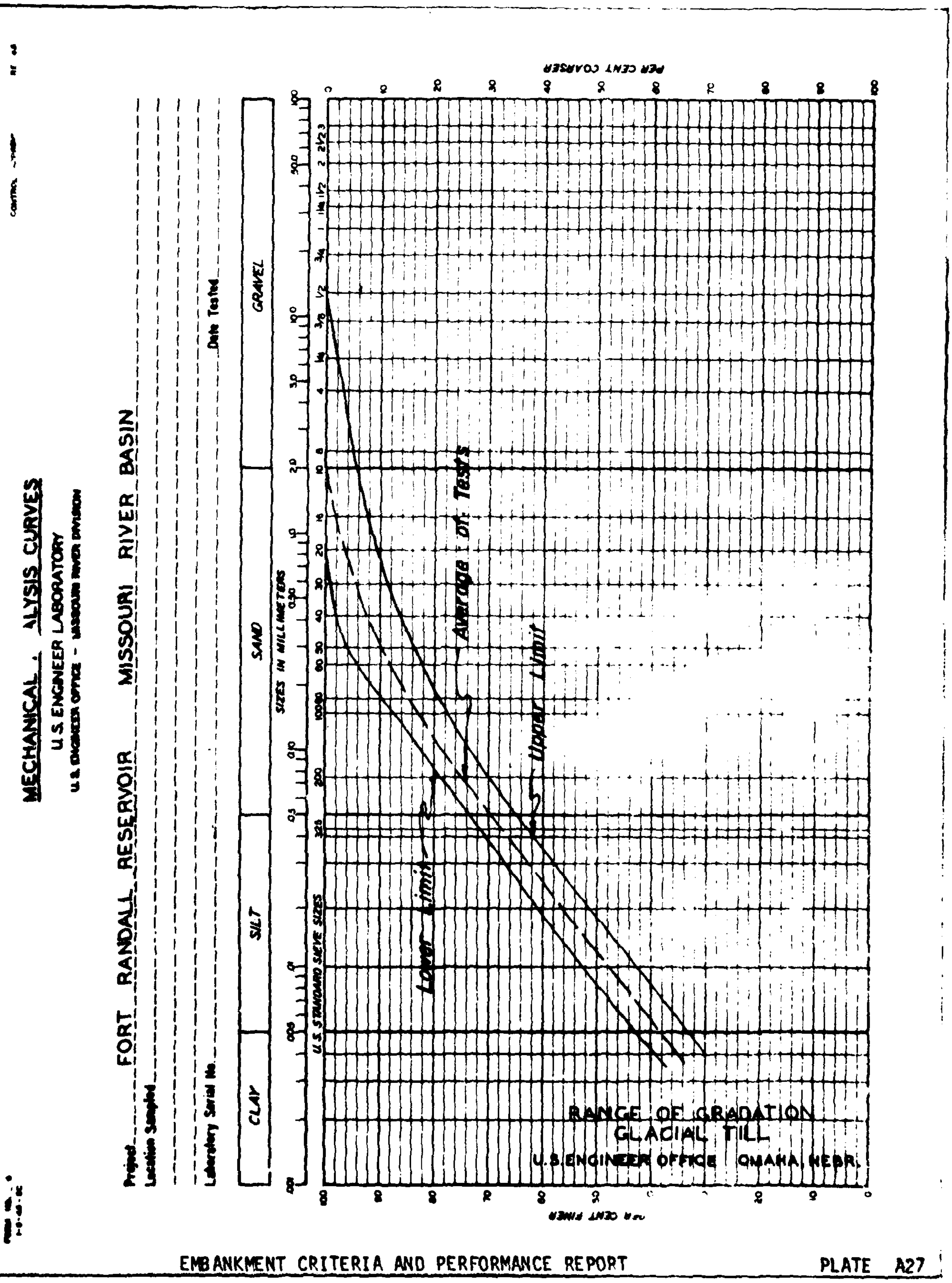
2

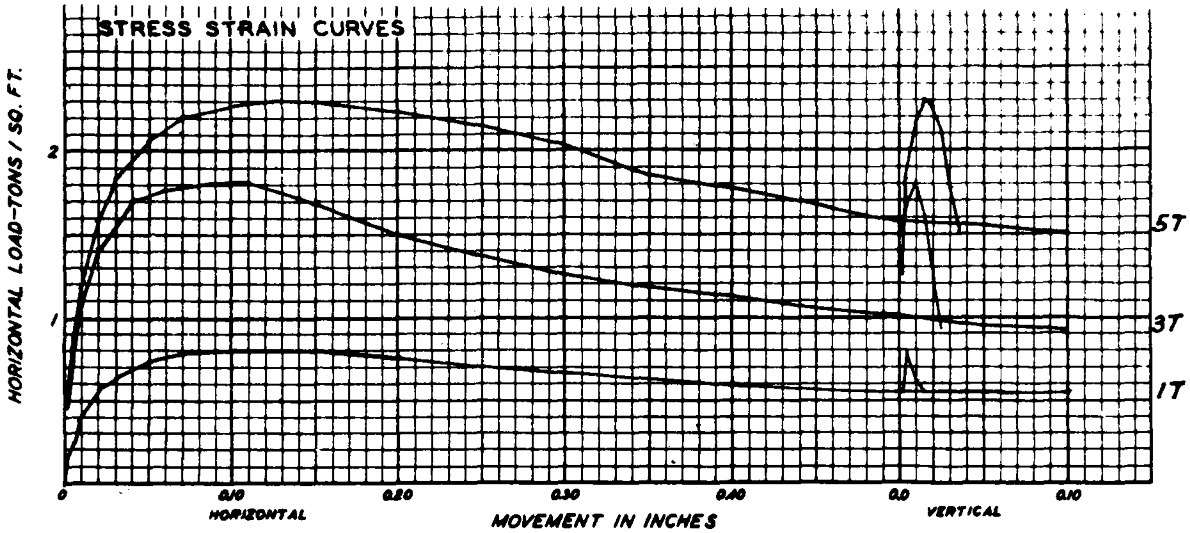

MECHANICAL ANALYSIS CURVES
U.S. ENGINEER LABORATORY
U.S. ENGINEER OFFICE - MISSOURI RIVER DIVISION
Project FORT RANDALL RESERVOIR MISSOURI RIVER BASIN
Location Sampled
Laboratory Serial No.
Date Tested
CLAY
SILT
SAND
GRAVEL
SIZES IN MILLIMETERS
U.S. STANDARD SIEVE SIZES
PER CENT FINER
PER CENT COARSER
Average of Tests
Upper Limit
Lower Limit
RANGE OF GRADATION
VALLEY SANDS
U.S. ENGINEER OFFICE OMAHA, NEBR.

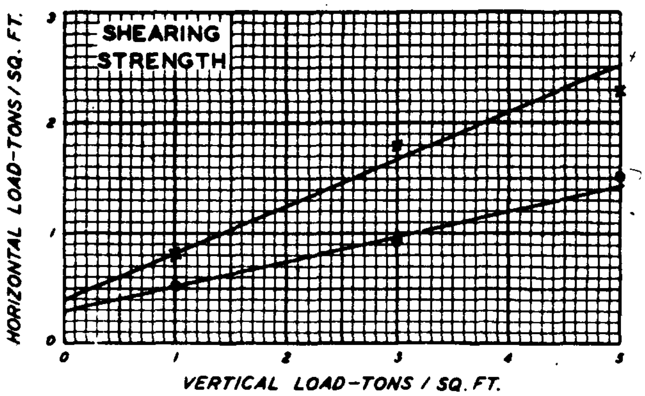


MECHANICAL ALYSIS CURVES
U.S. ENGINEER LABORATORY
U.S. ENGINEER OFFICE - MISSOURI RIVER DIVISION
Project FORT RANDALL RESERVOIR MISSOURI RIVER BASIN
Location Sampled
Laboratory Serial No.
Date Tested
CLAY
SILT
SAND
GRAVEL
SIZES IN MILLIMETERS
U.S. STANDARD SIEVE SIZES
PER CENT COARSER
PER CENT FINER
Lower Limit
Average of Tests
Upper Limit
RANGE OF GRADATION
RIGHT TERRACE LOESS
U.S. ENGINEER OFFICE OMAHA, NEBR.


MECHANICAL ANALYSIS CURVES
U.S. ENGINEER LABORATORY
U.S. ENGINEER OFFICE - MISSOURI RIVER DIVISION
Project FORT RANDALL RESERVOIR MISSOURI RIVER BASIN
Location Sampled
Laboratory Serial No.
Date Testing
CLAY
SILT
SAND
GRAVEL
SIZES IN MILLIMETERS
U.S. STAND'RD SIEVE SIZES
PER CENT FINER
Lower Limit
Average of Tests
Upper Limit
RANGE OF GRADATION
RIGHT TERRACE CLAYS
U.S. ENGINEER OFFICE OMAHA, NEBR.


MECHANICAL ANALYSIS CURVES
U.S. ENGINEER LABORATORY
U.S. ENGINEER OFFICE - MISSOURI RIVER DIVISION
FORT RANDALL RESERVOIR MISSOURI RIVER BASIN
Project
Location Sampled
Laboratory Serial No.
Date Tested
GRAVEL
SAND
SILT
CLAY
SIZES IN MILLIMETERS
U.S. STANDARD SIEVE SIZES
PER CENT COARSER
PER CENT FINER
Lower Limit
Average of Tests
Upper Limit
RANGE OF GRADATION
GLACIAL TILL
U.S. ENGINEER OFFICE OMAHA, NEBR.





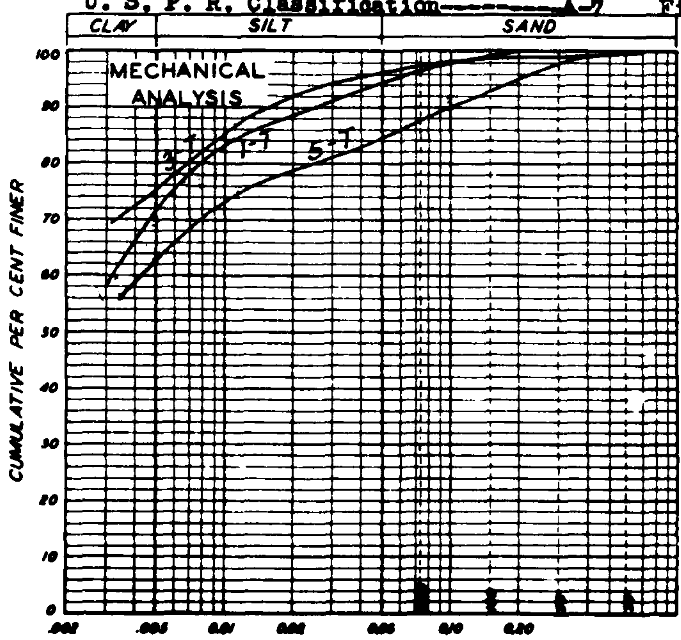
SHEARING STRENGTH

Maximum: c = .40 tan ϕ = .42
Ultimate: c = .30 tan ϕ = .22
Sample consolidated under vertical load before application of strain.
Strain applied by motor drive.

Moisture, before test = 29.5%
Voids, before test = 50.0%
Specific Gravity = 2.80

Liquid Limit———— 90
Plastic Limit———— 32
Plasticity Index———— 58
Flow Index———— 24
Shrinkage Limit———— 17.7%
Shrinkage Ratio———— 1.870
Lineal Shrinkage———— 18%
Field Moisture Equiv.—— 62%

Volumetric change from FME Moist.—83%
U. S. P. R. Classification———A-7

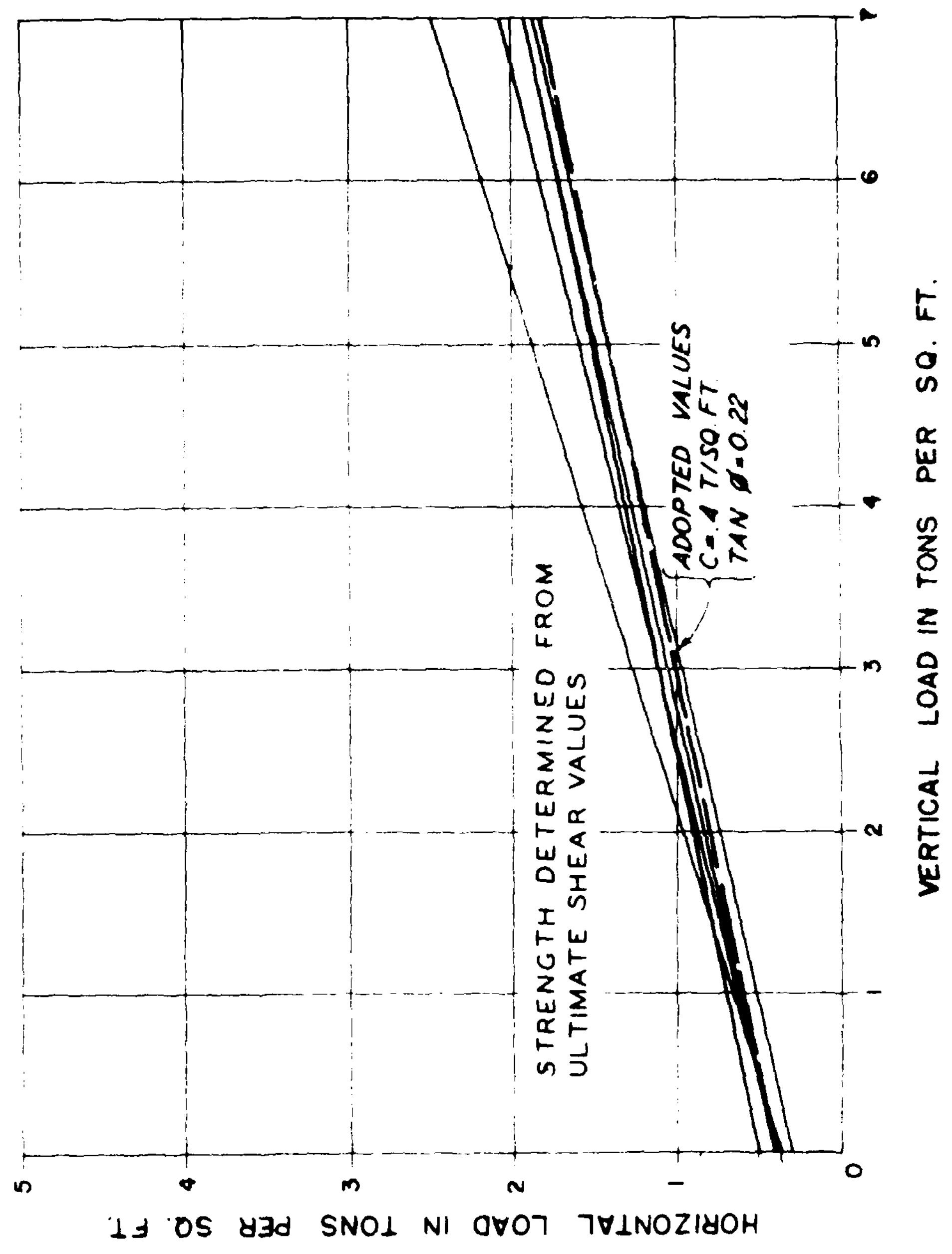


Project Ft. Randall
Stream Missouri River
Hole No. C-2
Location [redacted]
Ground Elev.
Sample No. U-6
Depth 46.0-46.7
Remarks

Insufficient material to run check test.
Dry weight, before test= 87.5 lbs /Cu.Ft.

RIGHT TERRACE CLAY
DIRECT SHEAR TEST

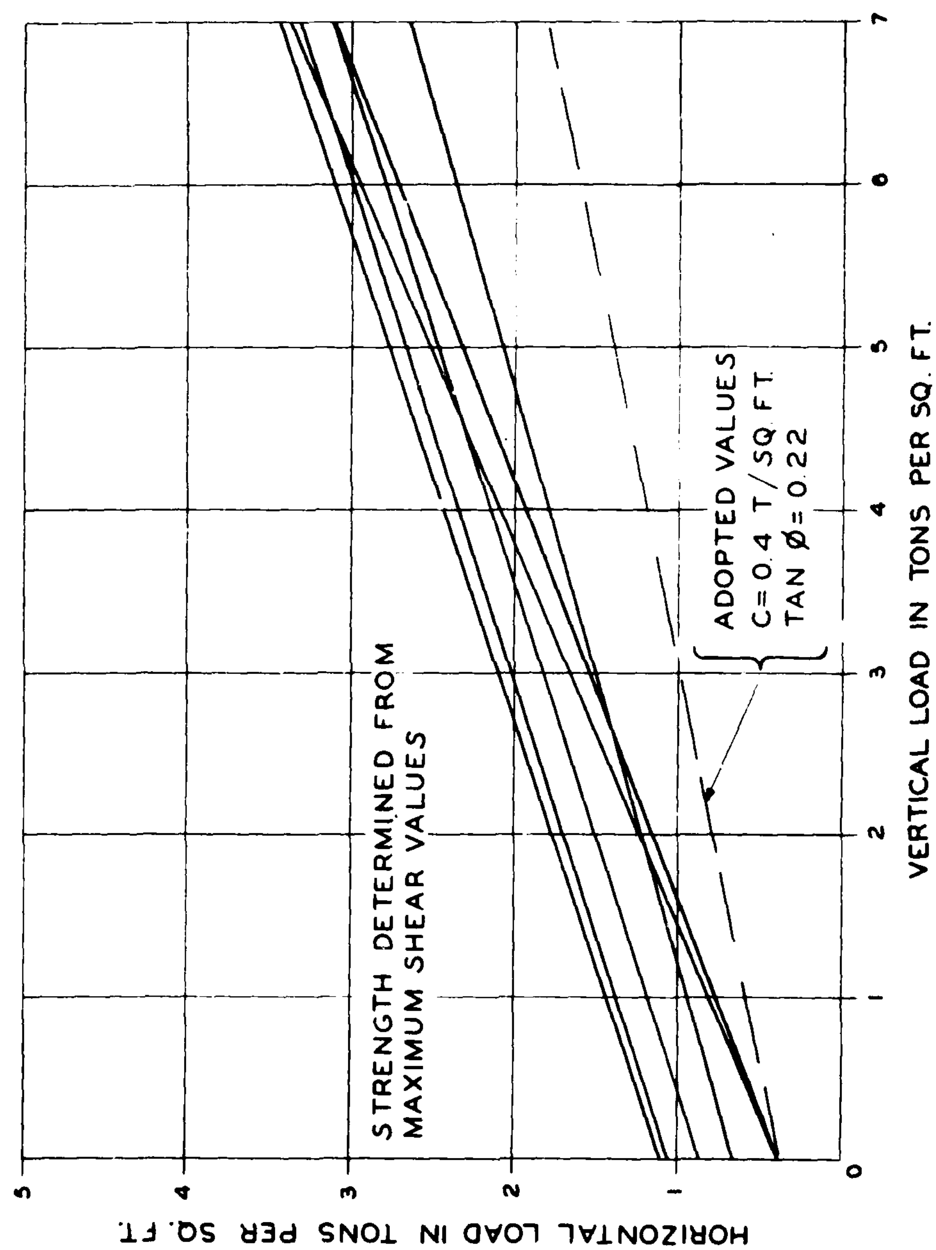
U.S. ENGINEER SOILS LABORATORY
U.S. ENGINEER OFFICE
MISSOURI RIVER DIVISION
KANSAS CITY, MO.



FT. RANDALL RESERVOIR
MISSOURI RIVER BASIN
SOUTH DAKOTA

GRAPHIC SUMMARY
DIRECT SHEAR TESTS
RIGHT TERRACE CLAY

U. S. ENGINEER OFFICE OMAHA, NEBR. APRIL 1946



FT. RANDALL RESERVOIR
MISSOURI RIVER BASIN
SOUTH DAKOTA

GRAPHIC SUMMARY
DIRECT SHEAR TESTS

RIGHT TERRACE CLAY

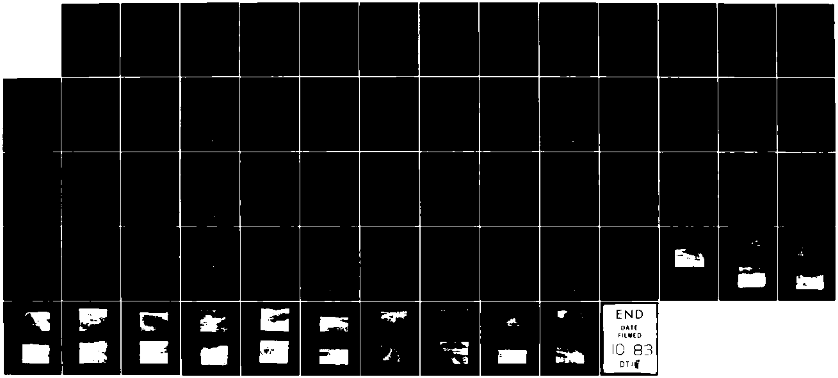
U. S. ENGINEER OFFICE OMAHA, NEBR. APRIL 1948.

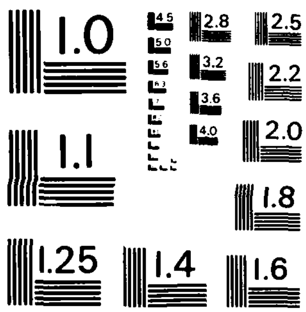
AD-A132 763 EMBANKMENT CRITERIA AND PERFORMANCE REPORT MISSOURI RIVER FORT RANDALL DAM - LAKE FRANCIS CASE(U) ARMY ENGINEER DISTRICT OMAHA NEBR MAR 83

2/2

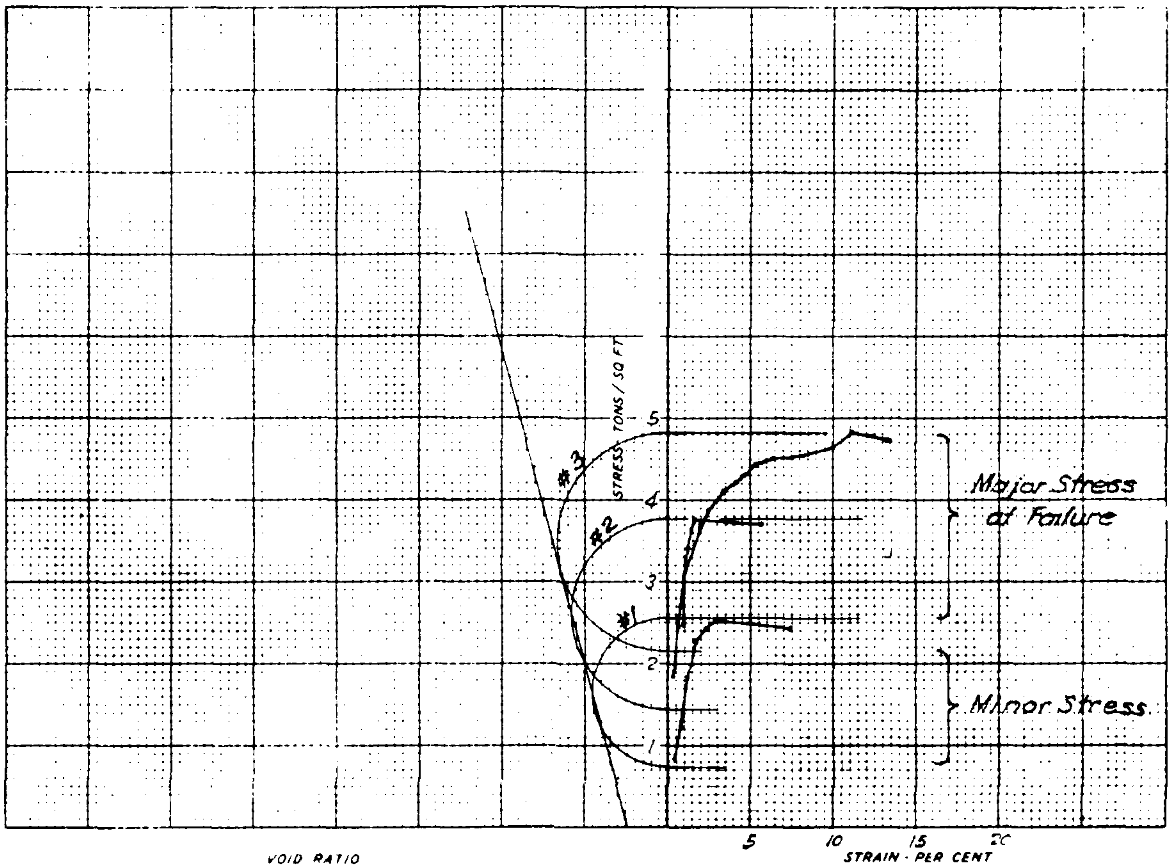
UNCLASSIFIED

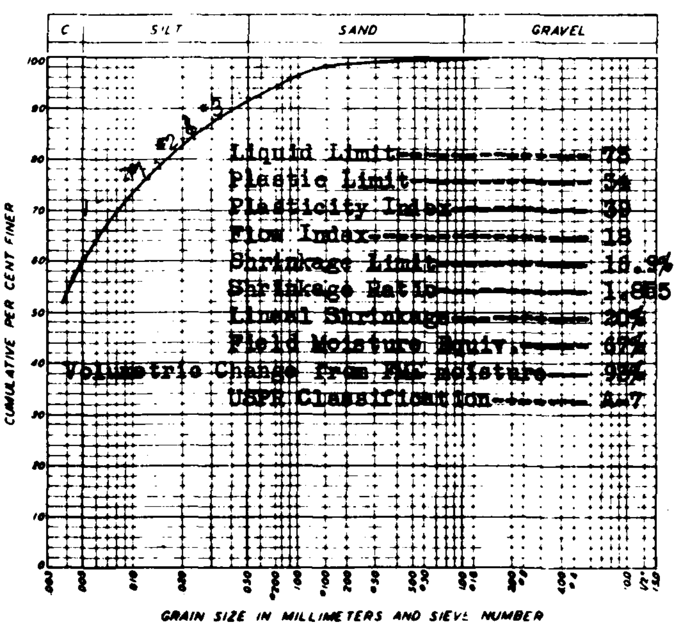
F/G 13/13 NL





MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A





Project Ft. Randall Dam

Stream Missouri River

Hole No C 2

Location [redacted]

Sample No U 11

Depth 72.7 - 74.4

Remarks c = .50 tan ø = .26

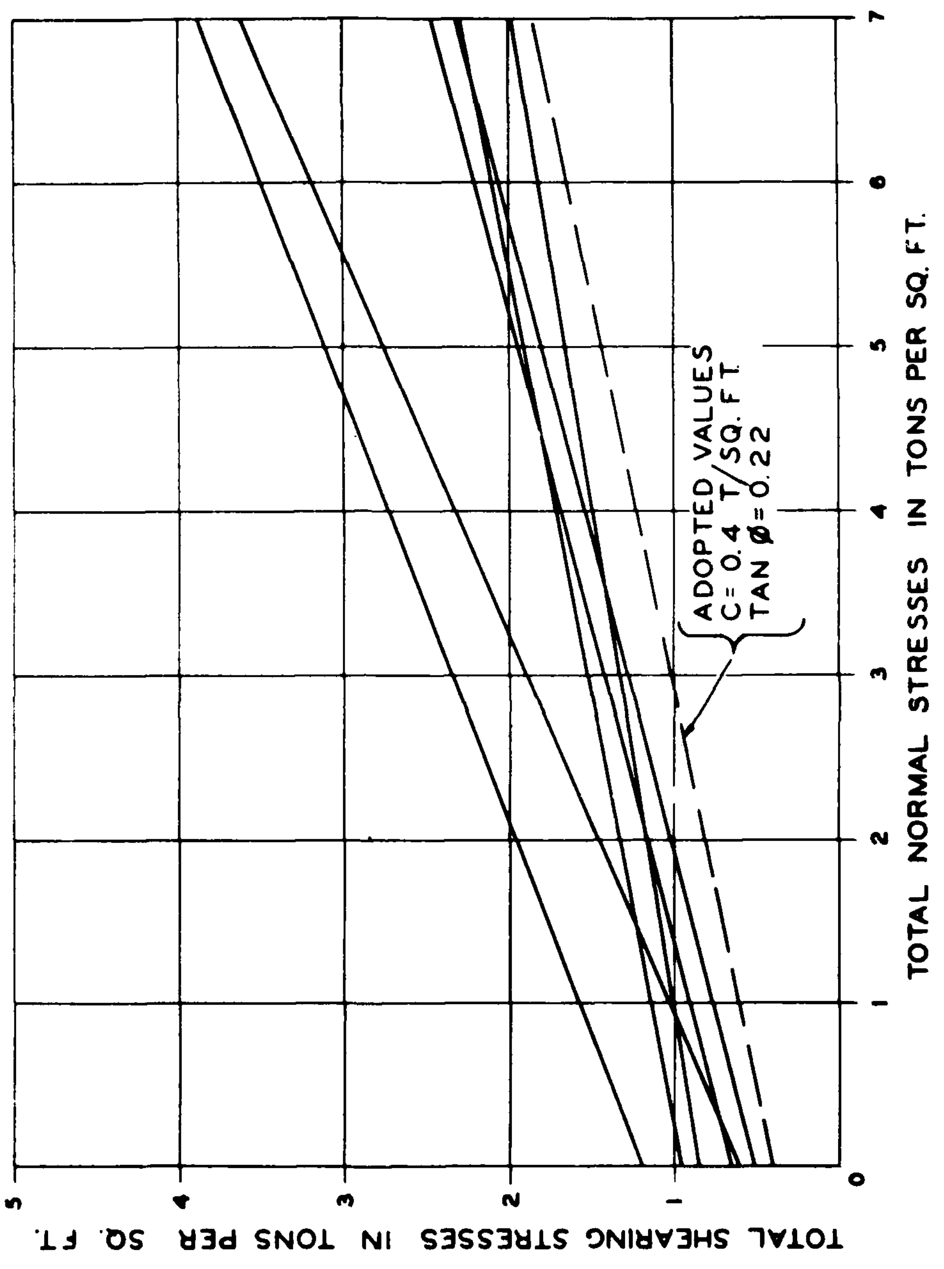
Moisture, before test = 32.8%

Dry weight, before test = 82.01bs. per cu.ft.

RIGHT TERRACE CLAY
TRIAXIAL
COMPRESSION TEST

U S ENGINEER SOILS LABORATORY

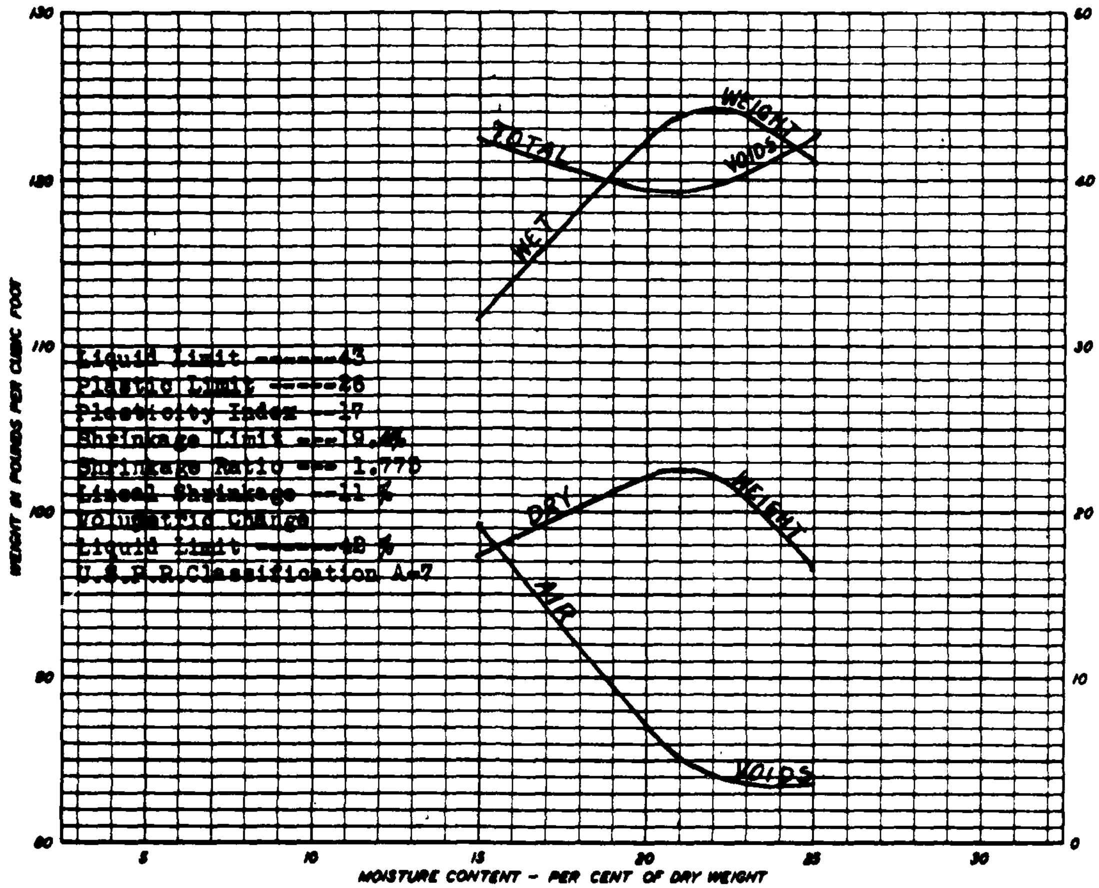
U S ENGINEER OFFICE
MISSOURI RIVER DIVISION
KANSAS CITY, MO

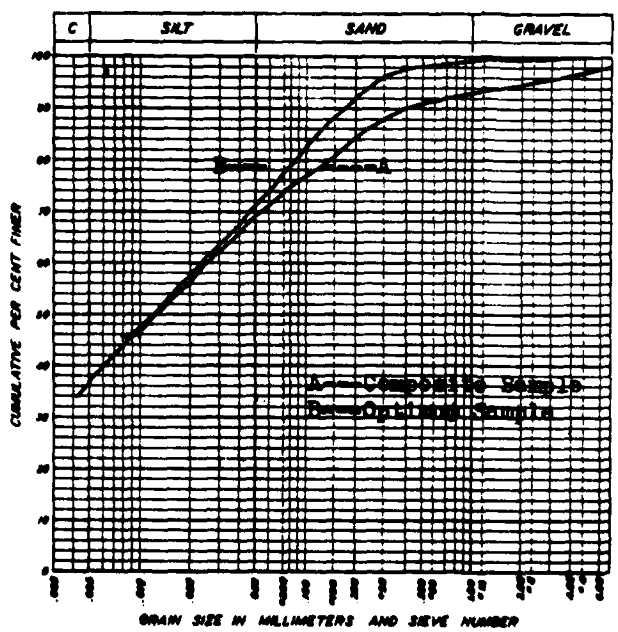


FT. RANDALL RESERVOIR
MISSOURI RIVER BASIN
SOUTH DAKOTA

GRAPHIC SUMMARY
TRIAXIAL COMPRESSION TESTS
RIGHT TERRACE CLAY

U. S. ENGINEER OFFICE OMAHA, NEBR. APRIL 1946.





Test No 238 Specific Gravity 2.70

Liquid Limit ________ Plastic Limit ________

STANDARD PROCTOR METHOD

Sample compacted in 1.5 inch layers by 25 blows of a 5.5 lb hammer dropped 12 inches.

Diameter of Cylinder 4 inches

Source of Sample Composite of samples 1 to 10 Depth 0 to 25.' Hole C42. Elevation-1434.9

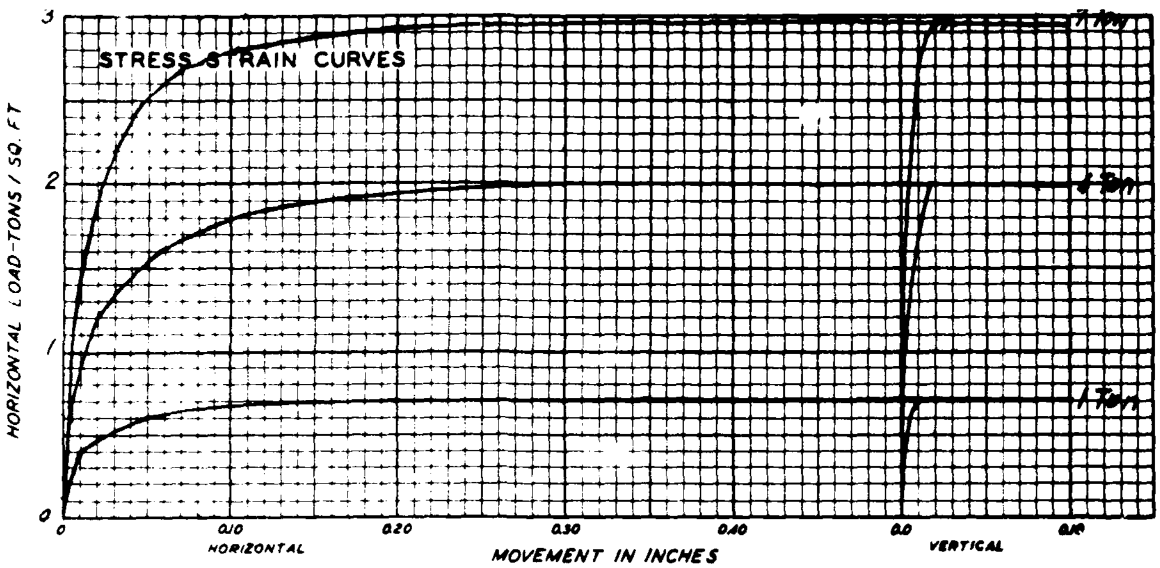
Project Fort Randall Damsite

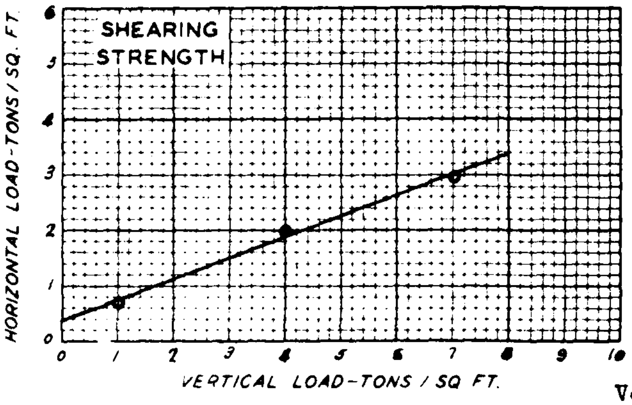
Stream Missouri River

GLACIAL TILL
OPTIMUM MOISTURE TEST

U. S. ENGINEER SOILS LABORATORY

U. S. ENGINEER OFFICE
MISSOURI RIVER DIVISION
KANSAS CITY MO.

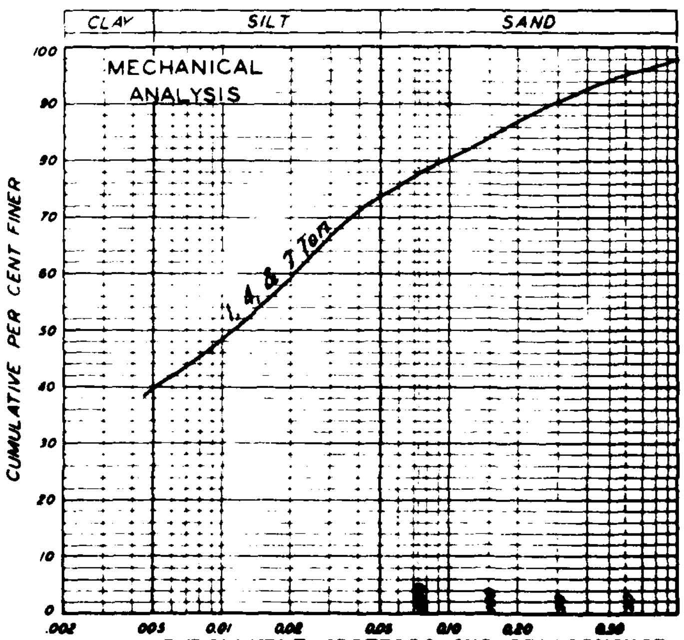




SHEARING STRENGTH
c = 0.40 tan ϕ = 0.37

Tests preconsolidated at vertical load.
Strain applied by motor drive:

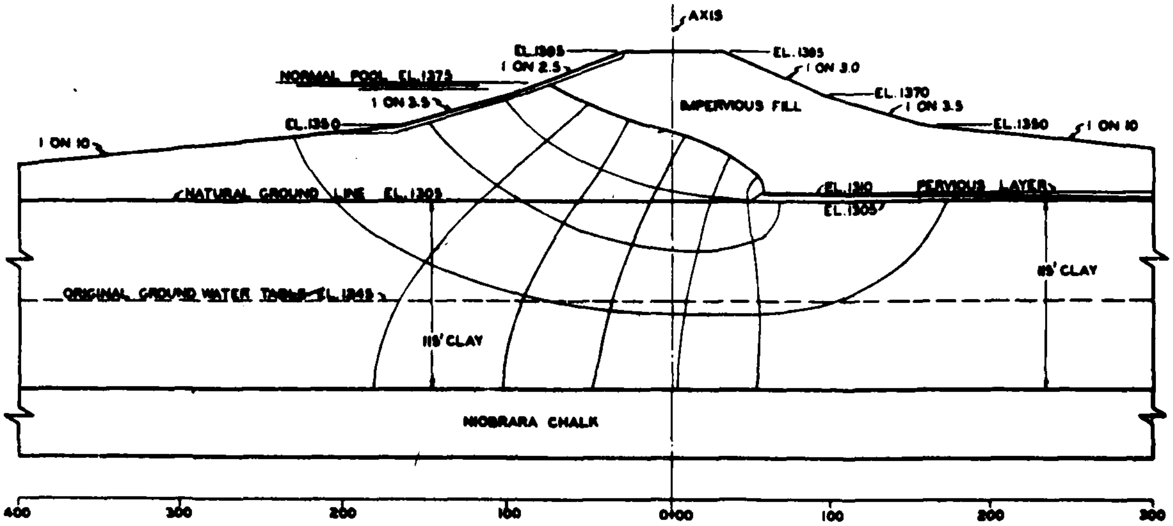
Moisture, before test ------20.9 %
Voids, before test ---------41.9 %
Dry Density ------98 lbs/cu. ft.
Specific Gravity ---------- 2.70
Liquid Limit --------------43
Plastic Limit ------------26
Plasticity Index -----------17
Shrinkage Limit -----------19.4 %
Shrinkage Ratio ----------- 1.775
Lineal Shrinkage -----------11 %
Volumetric Change from Liquid Limit-42 %
U. S. P. R. Classification -----A-7



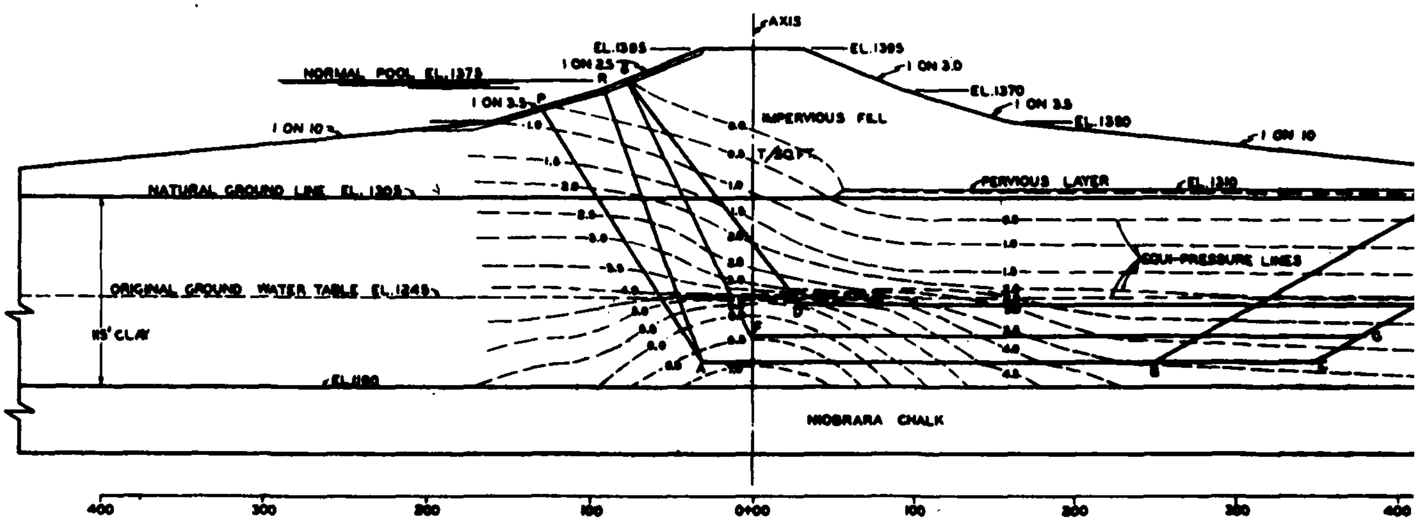
Project Fort Randall Damsite
Stream Missouri River
Hole No. C 42
Location
Ground Elev. 1434.9
Sample No. Composite of Samples 1 to 10
Depth 0-25'
Remarks Samples remolded.

RECOMPACTED GLACIAL TILL DIRECT SHEAR TEST

U.S. ENGINEER SOILS LABORATORY
U.S. ENGINEER OFFICE
MISSOURI RIVER DIVISION
KANSAS CITY, MO.



FLOW NET



TYPICAL RIGHT TERRACE SECTION THROUGH CLAY FOUNDATION SHOWING SLIDE PLANES ANALYZED

NOTES:
EQUI-PRESSURE LINES ARE OBTAINED FROM FLOW NETS.

DESIGN ASSUMPTIONS

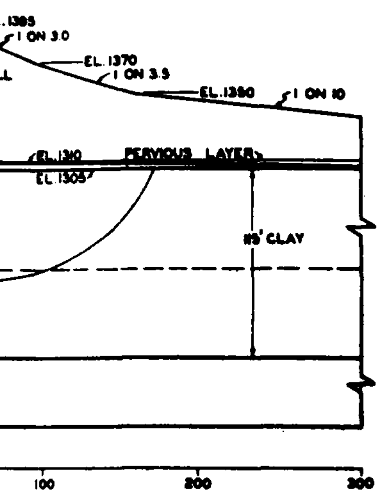
MATERIAL	DENSITY — TONS/CU.FT.		SHEARING STRENGTH	
	AS PLACED	SATURATED	C-TONS/SQ.FT.	TAN ϕ
ROLLED IMPERVIOUS FILL	.062	.064	0.35	0.35
CLAY FOUNDATION	IN PLACE .057	IN PLACE .059	0.40	0.22
DRAIN (PERVIOUS)	.052	.061	0	0.60

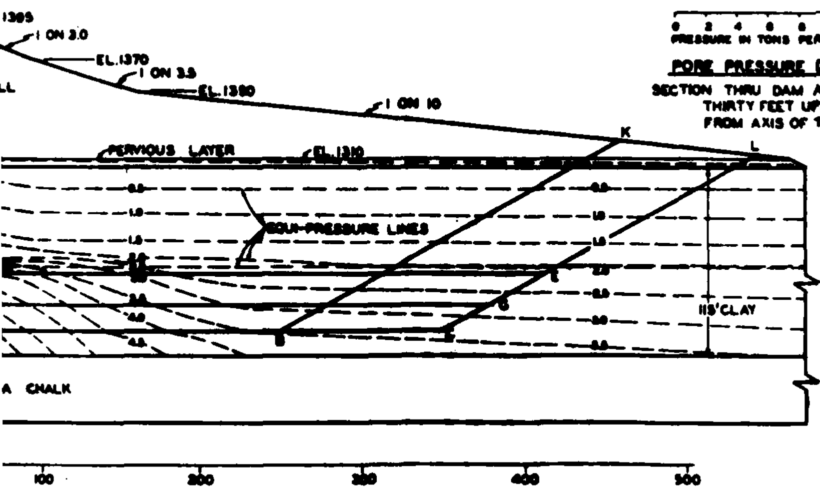
TRIAL	FACT[illegible] OF SAF[illegible]
PABK	3.[illegible]
PACL	2.[illegible]
RACL	3.[illegible]
● SDEL	2.[illegible]
SFGL	2.[illegible]

● CRITICAL SECTI[illegible]

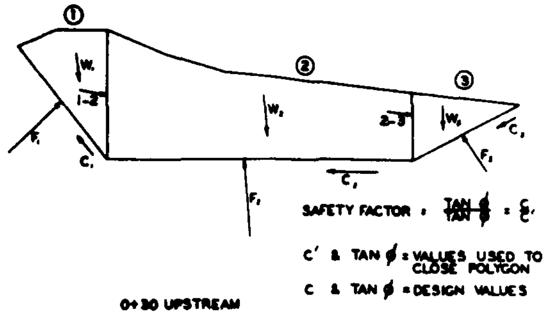
EMBANKMENT[illegible]

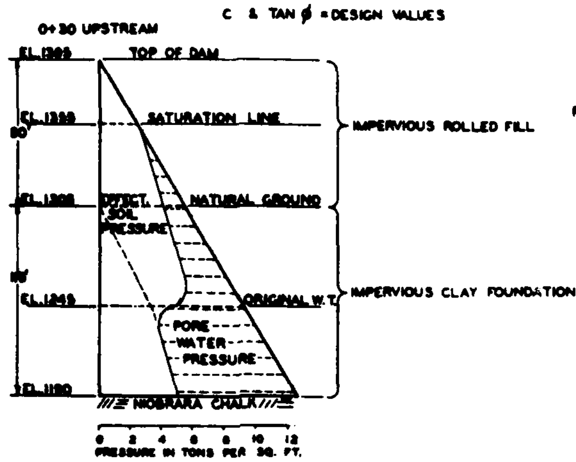
1





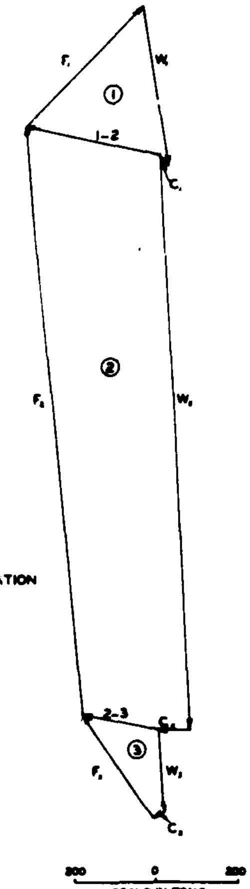
UNDATION SHOWING SLIDE PLANES ANALYZED





PORE PRESSURE DIAGRAM

SECTION THRU DAM AND FOUNDATION THIRTY FEET UPSTREAM FROM AXIS OF THE DAM



POLYGON OF FORCES
TRIAL SDEL
SAFETY FACTOR 2.2

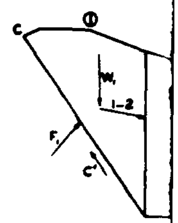
TIONS

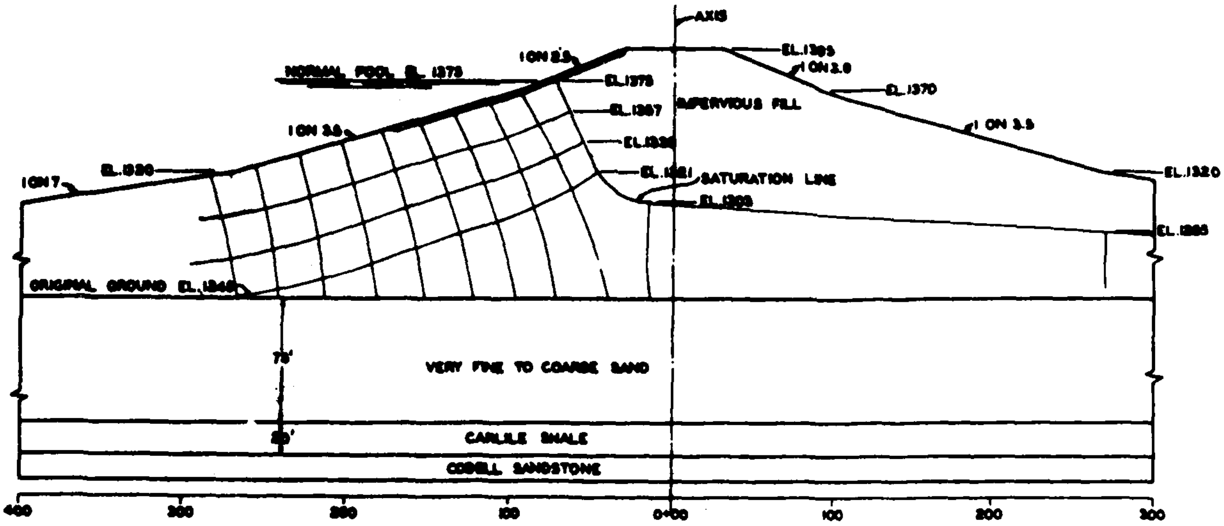
[illegible]	SHEARING STRENGTH	
[illegible]	C-TONS/SQ.FT.	TAN φ
[illegible]	0.35	0.36
[illegible]	0.40	0.22
[illegible]	0	0.60

TRIAL	FACTOR OF SAFETY
PASK	2.1
PACL	2.0
RACL	3.2
* SDEL	2.2
SFEL	2.5

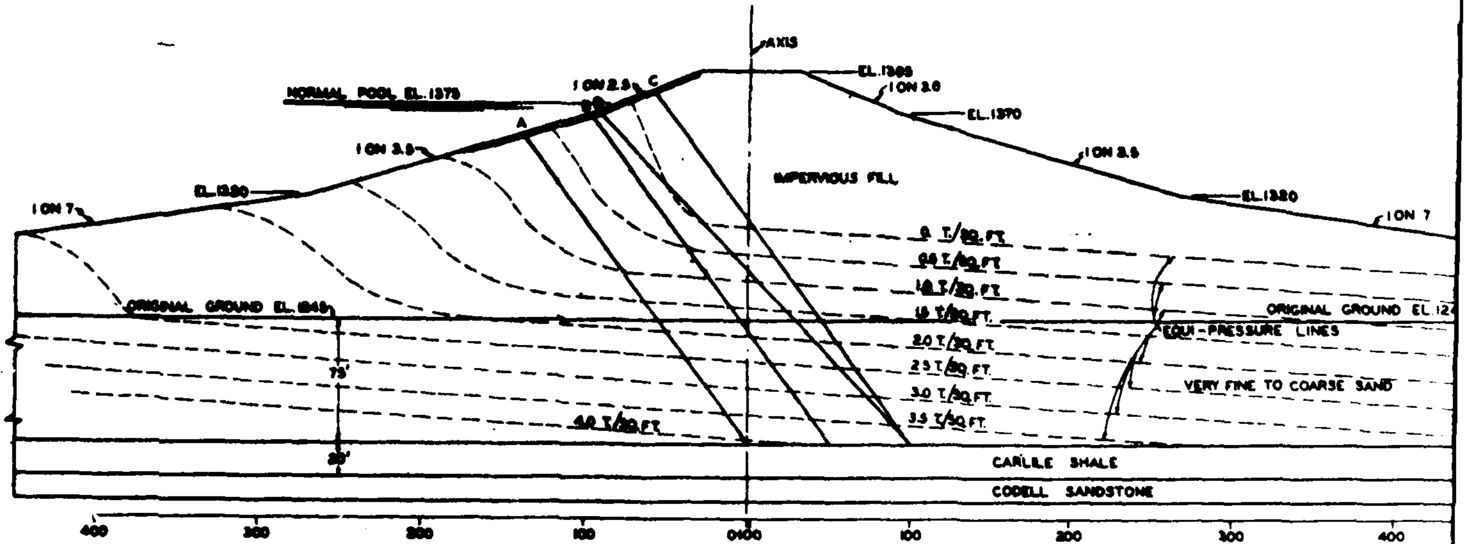
* CRITICAL SECTION

FORT RANDALL RESERVOIR
MISSOURI RIVER BASIN
SOUTH DAKOTA
SLIDE ANALYSIS
RIGHT TERRACE SECTION
CLAY FOUNDATION LAYER
U.S. ENGINEER OFFICE OMAHA, NEBR. APRIL 1946.





FLOW NET



TYPICAL VALLEY SECTION THROUGH CARLILE SHALE SHOWING SLIDE PLANES ANALYZED

NOTES:
EQUI-PRESSURE LINES ARE OBTAINED FROM FLOW NETS.

DESIGN ASSUMPTIONS

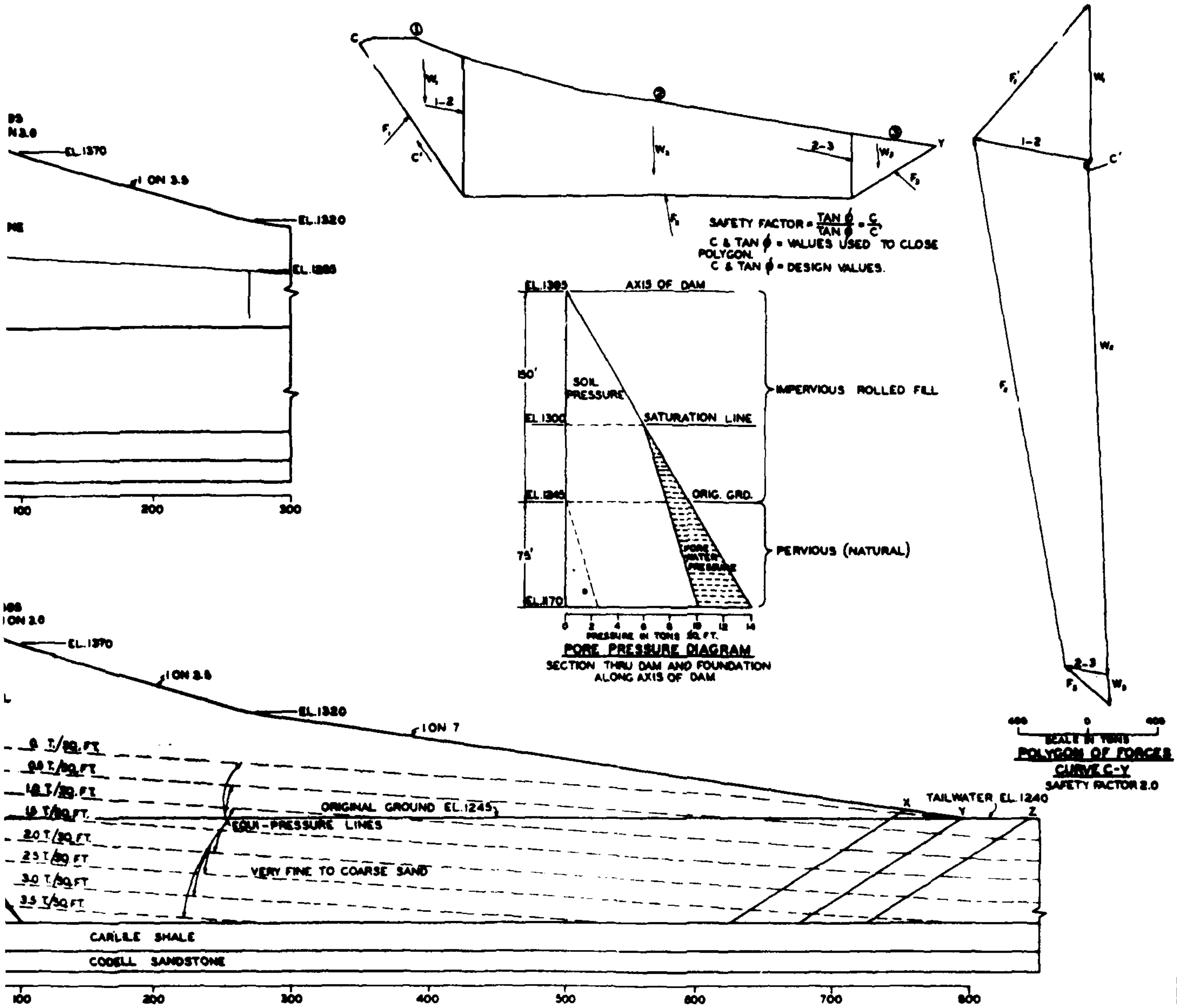
MATERIAL	DENSITY TONS/CU. FT.		SHEARING STRENGTH	
	AS PLACED	SATURATED	C- TONS/SQ.FT.	TAN Ø
ROLLED IMPERVIOUS FILL	.062	.064	0.35	0.35
PERVIOUS FOUNDATION	.052	.061	0.00	0.80
SAND-SHALE CONTACT-SHEARING STRENGTH: C=0 T/SQ FT. TAN Ø AS SHOWN IN TABLE "SUMMARY OF RESULTS"				

SUMMARY OF	
CURVE	REQUIR SAND-S ACT T SAFETY
	1.5
A-X	.116
B-X	.144
C-X	.164
C-Y	.16
B-Y	.162
C-Z	.16
D-Y	182

* CRITICAL SECT
ASSUMED COHESIO

1

EMBANKMENT



IALE SHOWING SLIDE PLANES ANALYZED

SUMMARY OF RESULTS			
CURVE	REQUIRED TAN ϕ AT SAND-SHALE CONTACT TO PROVIDE A SAFETY FACTOR OF 1.5	2.0	
A-X	.116	.232	
B-X	.144	.300	
C-X	.164	.340	
C-Y	.188	.358	•
B-Y	.162	.323	
C-Z	.188	.358	•
D-Y	.182	.358	

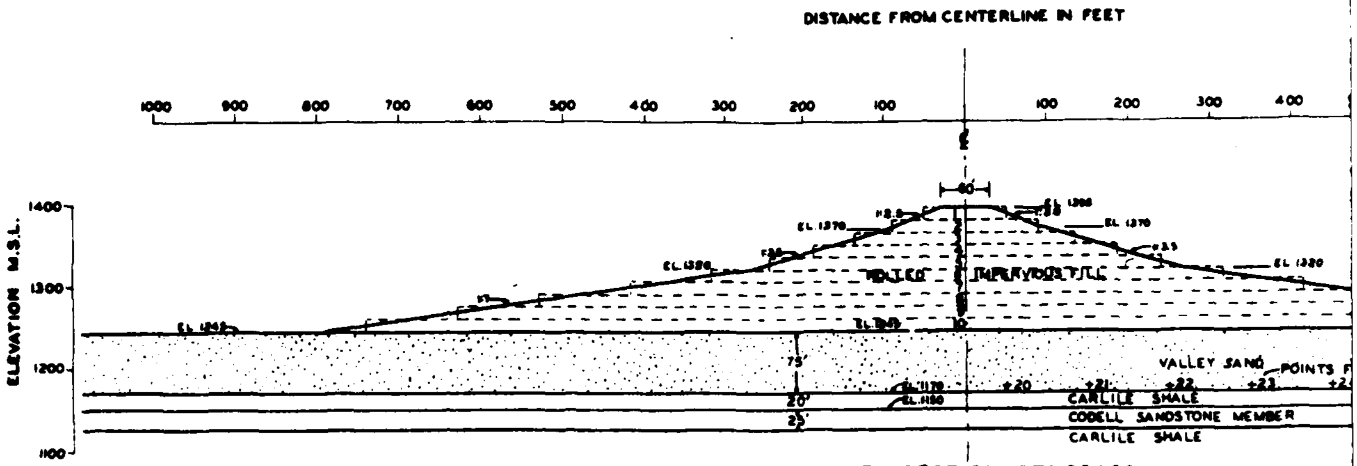
• CRITICAL SECTION

ASSUMED COHESION CONTACT LAYER C=0

FORT RANDALL RESERVOIR
MISSOURI RIVER BASIN
SOUTH DAKOTA
SLIDE ANALYSIS
VALLEY SECTION
SAND-SHALE CONTACT

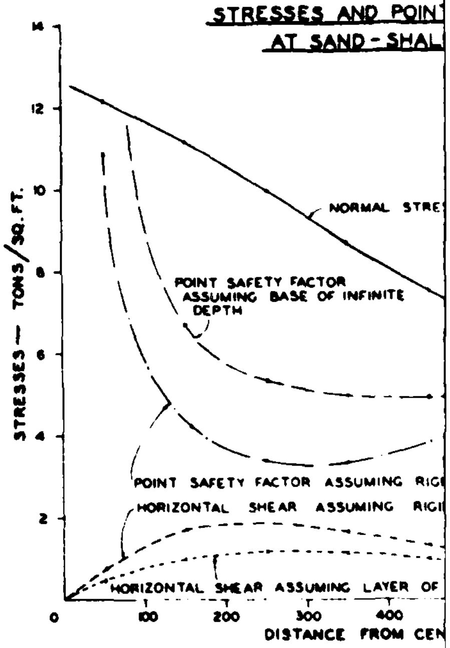
U.S. ENGINEER OFFICE OMAHA, NEBR. APRIL 1946.

2



VALLEY SECTION-STA 85+00

STRESSES AND POINT
AT SAND-SHAL

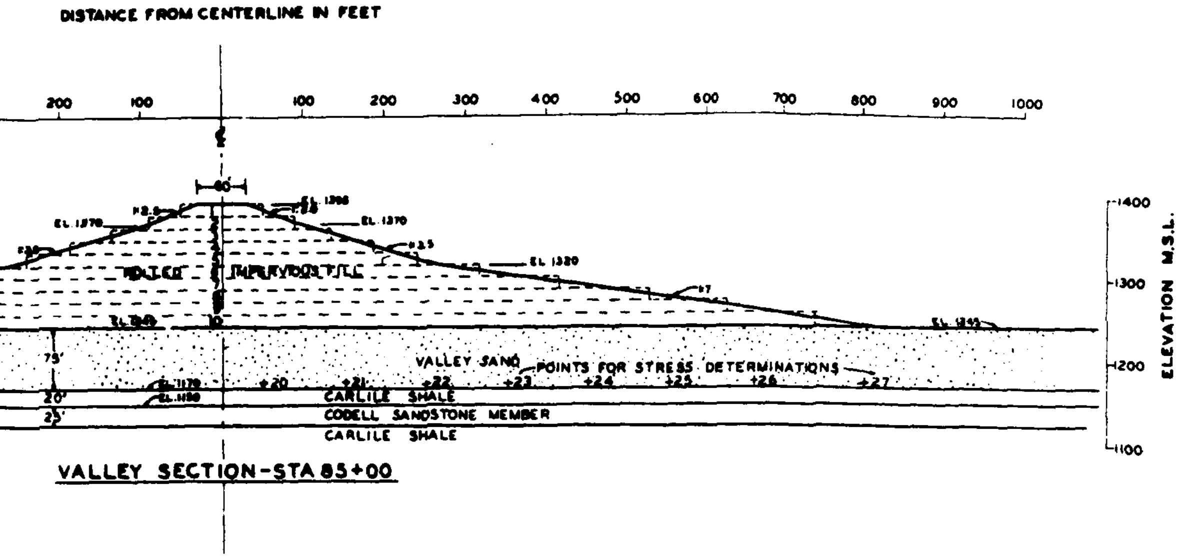


DESIGN ASSUMPTIONS

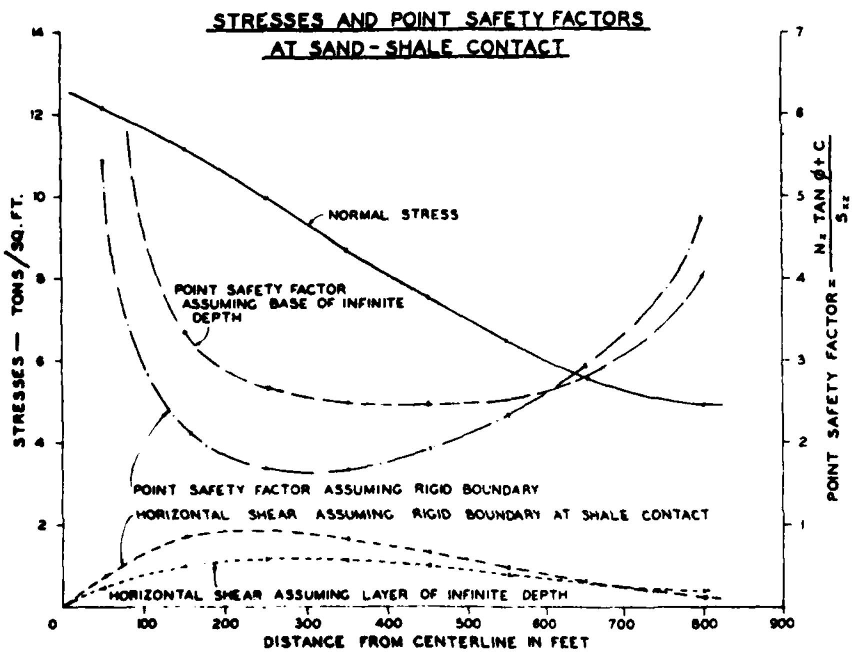
MATERIAL	DENSITY TONS/CU FT		SHEARING STRENGTH	
	AS PLACED	SATURATED	C TONS/SF	TAN Ø
ROLLED FILL	.062	.064	035	035
FOUNDATION SAND	IN PLACE .052	.061	0	06
SAND-SHALE CONTACT			0.1	0.3

EMBANKMENT CRIT

1



VALLEY SECTION-STA 85+00



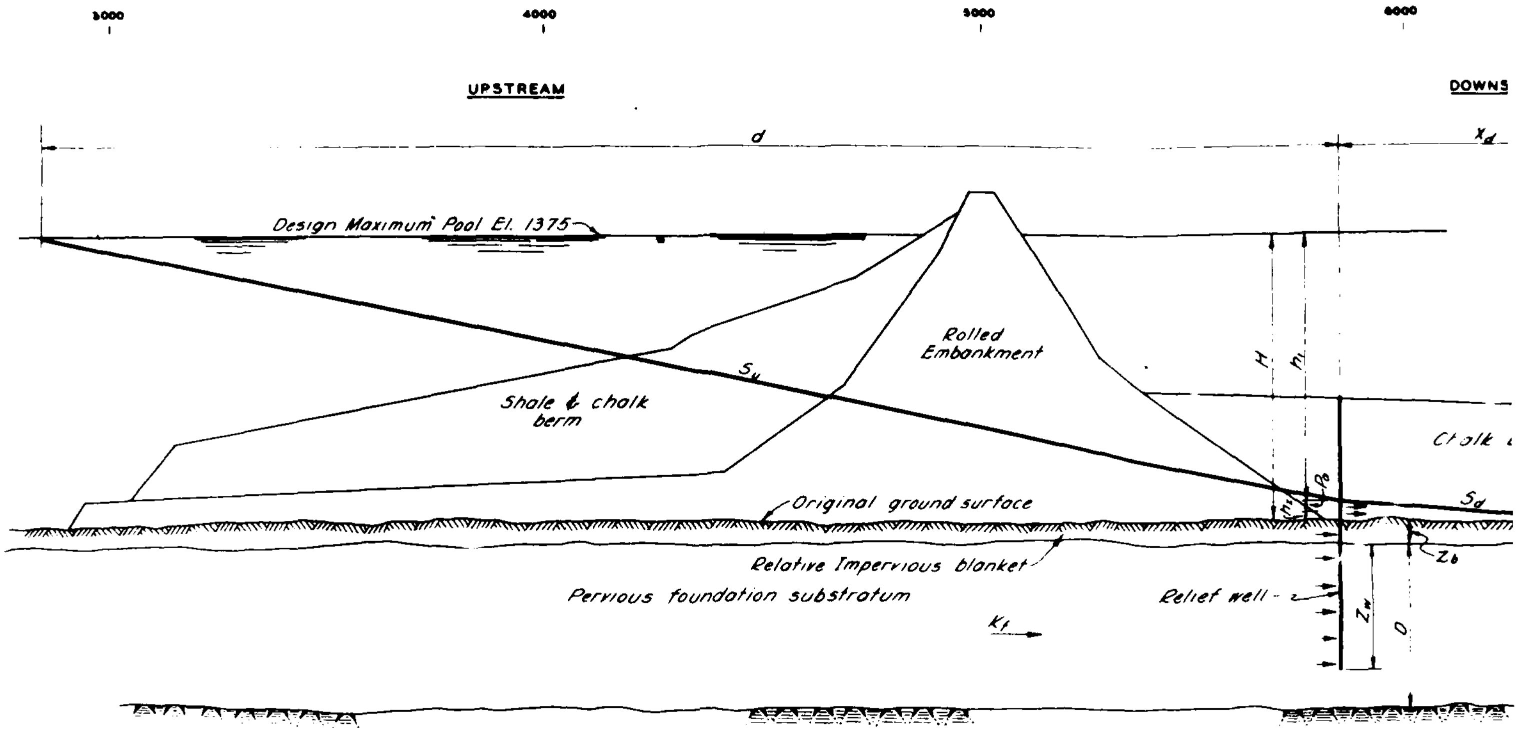
FORT RANDALL RESERVOIR
MISSOURI RIVER BASIN
SOUTH DAKOTA
STABILITY ANALYSIS
VALLEY SAND-SHALE CONTACT
ELASTIC THEORY METHOD
U.S. ENGINEER OFFICE OMAHA NEBR APRIL 1946

2

SUMMARY OF RELIEF WELL SPACING & DISCHARGE COMPUTATIONS

	Stations 61+50 to 65+00	65+00 to 75+00	75+00 to 81+00	81+00 to 85+00	85+00 to 95+00
Orig. G.S.Elev.	1253	1235	1238	1247	1248
Ave. Bedrock Elev.	1170	1165	1080	1100	1160
Ave. Well Bottom Elev.	1170	1165	1150	1150	1160
Total Head, $H^{(1)}$	122	140	137	126	127
Thickness of Substratum, D	78	65	153	142	83
Depth of Well, Z_w	Full.Pene.	Full Pene.	83	92	Full Pene.
Well Spacing, a	120	100	70	80	100
Upstream Head Loss, h_1	118.0	136.3	131.6	123.2	123.4
Upstream Gradient, S_r	.0369	.0426	.0411	.0385	.0386
Downstream Gradient, S_1	.0100	.0093	.0135	.0120	.0090
Net Gradient, S	.0269	.0333	.0276	.0265	.0286
Well Discharge, Q_w(cfs)	.50	.43	.59	.60	.47
Head Loss out of Screen, h_x	2.06	1.85	2.30	2.32	1.97
Mean potential, P_a	1.87	1.83	3.14	2.46	1.57
Midpoint potential P_m	2.23	2.20	1.95	1.92	1.88
Total uplift, $P_m + h_x$	4.29	4.05	4.25	4.24	3.85

Notes:
1. See Plate A-39 for definition of symbols
2. Midpoint potential equals $P_a + h_3$ for full penetration well.
3. Upstream Resistance d = 3200
4. Downstream Resistance x_3 = 400'
5. Radius of Relief Well - 0.5'
6. Design pool elevation - 1375 msl.



TYPICAL SECTION ILLUSTRATING NOMENCLATURE

TYPICAL COMPUTATION FOR FULLY PENETRATING WELL

STA. 65+00 TO STA. 75+00

DESIGN DATA

Average original ground surface elevation = 1235.
Average bedrock elevation = 1165.
H = 140 ft
D = 65 ft
Full penetrating well formula = $P_a = \left(\frac{a}{2\pi} \log_e \frac{a}{2\pi r_w}\right) S = F S$

Assume well spacing "a" and value for $P_a + h_x$ and compute total uplift by trial and error. Total uplift $(P_a + h_x + h_3) \leqq$ Allowable uplift.

Assume a = 100

		1st Trial	2nd Trial
$P_a + h_x$ (Assumed)	=	3.2	3.7
h_1	=	136.8	136.3
S_r	=	.0428	.0426
S_1	=	.0080	.0093
S	=	.0348	.0333
Q_w (in c f s)	=	.45	.43
h_x	=	1.95	1.89
$P_a = FS = 55S$	=	1.91	1.83
$P_a + h_x$	=	3.86	3.72
h_3	=	.38	.37
$h_x + P_a + h_3$	=	4.24	4.09

The computed value for $P_a + h_x$ corresponds to the assumed values in the second trial and the total computed potential, $h_x + P_a + h_3$ is slightly less than the allowable value, therefore the well spacing of 100 ft. is adequate.

TYPICAL COMPUTATION FOR PARTIALLY PENETRATING WEL

STA. 75+00 TO STA. 81+00

DESIGN DATA

Average ground surface elevation = 1238
Average bedrock elevation = 1080
Bottom of well elevation = 1150
H = 137 ft
D = 153 ft - Transformed D' = 612 ft
Z_w = 83 ft - Transformed Z'_w = 332 ft

Mean potential formula for partially penetrating wells

$$P_a = \left[\frac{aD'}{2\pi Z'_w} \log_e \frac{a}{2^{(\frac{D}{Z_w}-1)(1-\frac{Z_w}{D})}\pi r_w}\right] S$$

Mid-point potential formula for partially penetrating wells

$$P_m = \left[\frac{aD'}{2\pi Z'_w} \log_e \frac{a}{2^{(1-\frac{Z_w}{D})}\pi r_w}\right] S$$

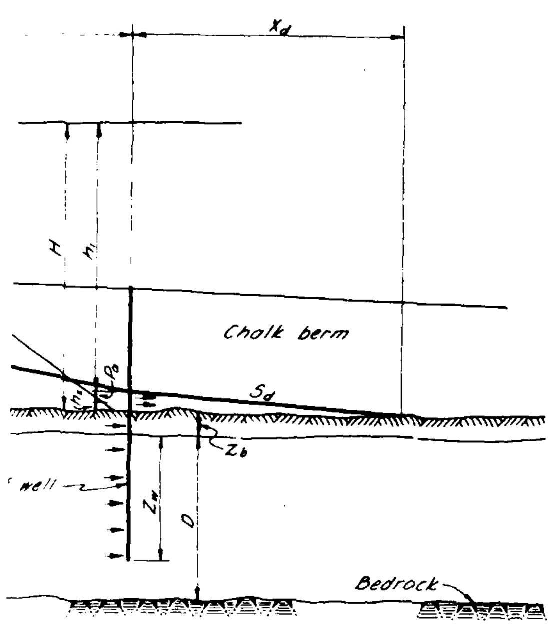
Assume well spacing "a" and value for $P_a + h_x$ and compute total uplift, using the transformed depths and permeability, by trial and error. Total uplift $P_m + h_x \leqq$ allowable uplift.

		1st Trial	2nd Trial	3rd Trial
a (Assumed)	=	100	100	70
$P_a + h_x$ (Assumed)	=	6.0	6.4	5.4
h_1	=	131.0	130.6	131.6
S_r	=	.0409	.0408	.0411
S_1	=	.0150	.0160	.0135
S	=	.0259	.0248	.0276
Q_w	=	.79	.76	.59
h_x	=	2.79	2.71	2.30
P_a	=	3.83	3.70	3.14
$P_a + h_x$	=	6.62	6.41	5.44
P_m	=		2.80	1.95
$P_m + h_x$	=		5.51	4.25

The computed value for $P_a + h_x$ corresponds to the assumed value in the second trial however the total computed potential, $P_m + h_x$, is greater than the allowable value, therefore the spacing of 100 ft is inadequate. The computed value for $P_a + h_x$ corresponds to the assumed value in the third trial and the total computed potential $P_m + h_x$ is approximately equal to the allowable value, therefore the spacing of 70 ft. is adequate.

6000

DOWNSTREAM



NOMENCLATURE

- Z_b = Thickness of relatively impervious downstream blanket.
- D = Thickness of pervious substratum.
- Z_w = Depth of proposed relief well.
- H = Total head
- P_o = Mean potential over plane of wells.
- P_w = Surface potential at mid-point between partially penetrating wells.
- h_x = Head loss due to flow out of well screen into chalk berm.
- h_1 = $H-(P_o+h_x)$ = Total headloss from source to line of wells.
- d = Effective resistance upstream of line of wells.
- X_d = Effective resistance downstream of line of wells.
- S = $S_u - S_d$ = Net potential gradient producing discharge from wells.
- a = Well spacing.
- h_3 = $0.11aS$ = Potential midway between fully penetrating wells in excess of the average potential over the plane
- K_f = Horizontal permeability of pervious substratum.
- Q_w = $K_f DaS$ = Discharge of well.
- r_w = Radius of proposed well.

...LLY PENETRATING WELL

...81 + 00

= 1238
= 1080.
= 1150.

ft
ft

...y penetrating wells.

...] S

...rtially penetrating wells

...for $P_o + h_x$ and compute
...' depths and perme-
...uplift $P_w + h_x \overline{\overline{\leq}}$ allow-

...ND Trial	3RD Trial
100	70
6.4	5.4
130.6	131.6
.0408	.0411
.0160	.0135
.0248	.0276
.76	.59
2.71	2.30
3.70	3.14
6.41	5.44
2.80	1.95
5.51	4.25

...sponds to the assumed
...the total computed potential,
...able value, therefore
...iate. The computed value
...ned value in the third trial
...$P_w + h_x$ is approximately
...efore the spacing of 70 ft.

GENERAL DESIGN ASSUMPTIONS & DATA

1. d = 3200 ft. All gradients are approximately equal.
2. X_d = 400 ft as indicated by effective radius of well by pumping test.
3. K_f = .002 c f s = Horizontal permeability of previous substratum.
4. Ratio of horizontal permeability to vertical permeability of foundation material is in the order of 16 to 1
5. In the case of partial penetrating wells the depth of the well and depth of the substratum is transformed in accordance with the item 4 above or $\sqrt{K_f/K_v} = \sqrt{16/1} = 4$
6. Weight of saturated blanket material = 115 lb/cu ft therefore the allowable uplift equals 0.84 Z_b for factor of safety of 1.0
7. Assume uniform average downstream blanket thickness of 5 feet across entire valley. Therefore allowable uplift = 0.84 x 5 = 4.2 ft at all locations.
8. Head loss for flow out of well screen = $h_x = (5.89\,Q_w)^{[illegible]}$.
9. Design maximum pool water surface elevation = 1375.
10. Assume radius of well - r_w = 0.5 ft. Actual inside diameter of well screen is only 8 inches however it is assumed the effective radius is increased due to the gravel pack
11. All wells with exception of those in deepest channel are fully penetrating. Those in the deep channel penetrate to a minimum elevation of 1150.

MISSOURI RIVER

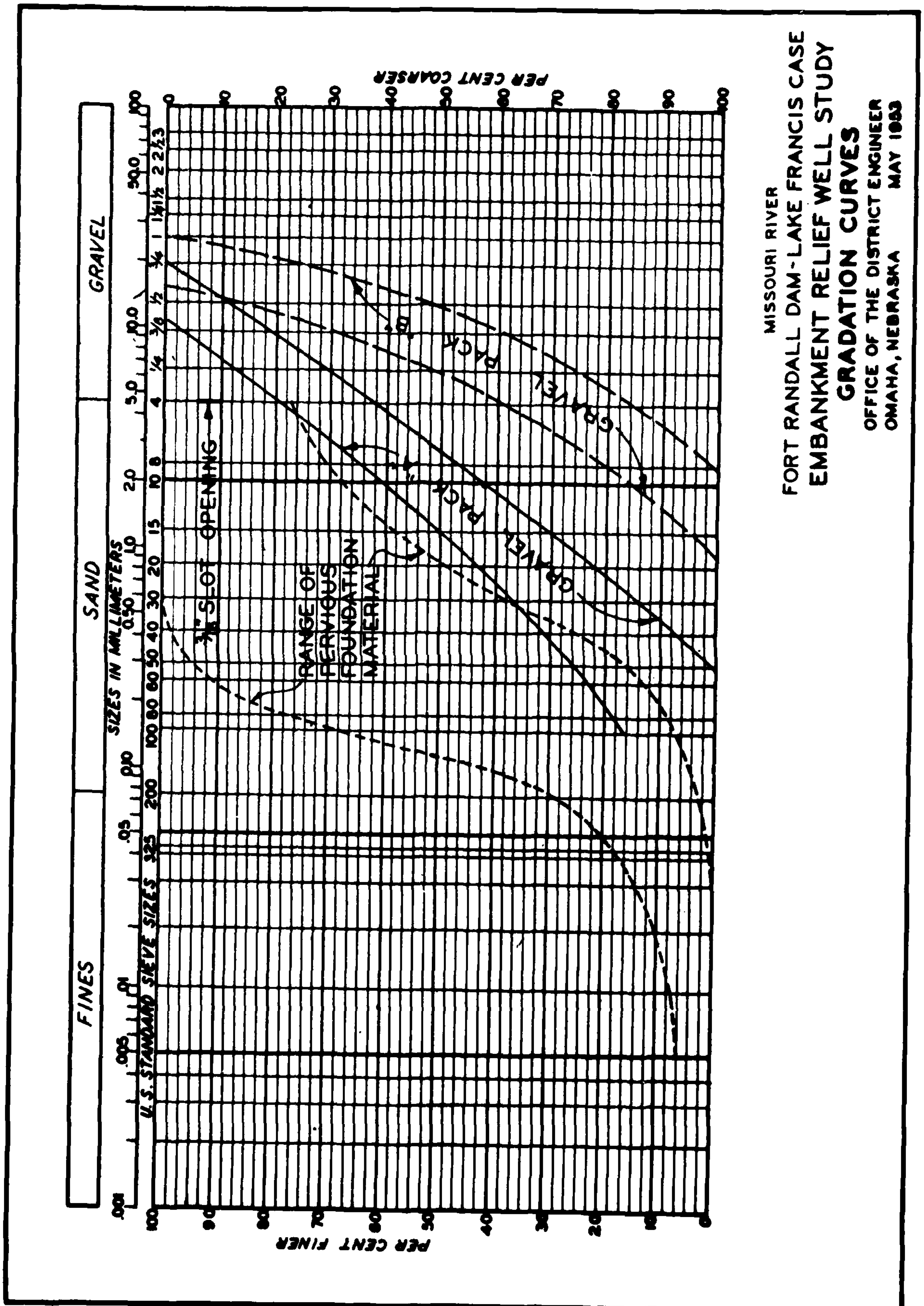
FORT RANDALL DAM-LAKE FRANCIS CASE

EMBANKMENT RELIEF WELL STUDY

TYPICAL WELL SPACING COMPUTATIONS

OFFICE OF THE DISTRICT ENGINEER

OMAHA NEBRASKA MAY 1953


MISSOURI RIVER
FORT RANDALL DAM-LAKE FRANCIS CASE
EMBANKMENT RELIEF WELL STUDY
GRADATION CURVES
OFFICE OF THE DISTRICT ENGINEER
OMAHA, NEBRASKA
MAY 1953
GRAVEL
SAND
FINES
SIZES IN MILLIMETERS
U.S. STANDARD SIEVE SIZES
PER CENT FINER
PER CENT COARSER
GRAVEL PACK B
GRAVEL PACK
RANGE OF PERVIOUS FOUNDATION MATERIAL
3/16" SLOT OPENING

FORT RANDALL DAM, PERMANENT RECORD SAMPLES (ROLLED EMBANKMENT)

Box Sample No.	Station	Range	Elevation	Soil Classification	Mechanical Analysis GR. %	SA. %	FI %	LL	PI	S. G.	M. C. %	Dry Density PCF	R
A. Initial Earthwork:													
1	95+90	4337	1257.5	Sandy Clay	0	28	72			2.69	17.9	96.9	
2	94+87	5218	1261.5	Sandy Clay	0	36	64			2.70	17.5	108.6	
3	94+50	4955	1268.3	Sandy Clay	0	33	67			2.69	16.2	108.4	
4	88+00	4890	1254.2	Sandy Clay	0	33	67			2.70	19.3	105.8	
5	84+17	5250	1252.1	Sandy Clay	0	32	68			2.74	19.0	104.1	
7	99+95	4595	1275.3	Sandy Clay	3	24	73			2.71	21.7	100.1	
8	100+27	4978	1283.8	Sandy Clay	20	22	58			2.71	19.9	104.3	
10	95+50	5200	1286.0	Clayey Sand	4	56	40			2.70	17.4	98.2	
11	94+12	4798	1285.5	Clayey Sand	23	33	44			2.66	20.1	101.3	
12	88+00	5142	1268.6	Clayey Sand	6	46	48			2.74	14.0	100.3	
13	102+43	4925	1319.1	Sandy Clay	7	27	68			2.74	19.9	97.4	
14	93+20	4936	1299.0	Sandy Clay	3	44	53			2.70	20.5	99.0	
15	95+00	5075	1316.8	Sandy Clay	3	27	70			2.69	20.6	103.0	
16	98+50	4900	1342.7	Sandy Clay	2	23	75			2.75	21.3	99.3	
17	110+10	5025	1363.6	Sandy Clay	0	32	68			2.71	20.2	100.0	
18	85+00	4675	1269.6	Sandy Clay	3	26	71			2.69	21.1	101.3	
19	2+48	3+00	1336.5	Sandy Clay	3	22	75			2.70	20.8	100.3	
20	110+05	4940	1365.7	Sandy Clay	0	22	78			2.68	21.8	100.6	
B. Earthwork Stage II:													
1	90+00	4700	1281.2	Sandy Clay	7	18	75	47	26	2.67	19.8	93.0	C
2	87+45	4895	1279.8	Sandy Gr. Clay	23	20	57	44	22	2.72	19.2	99.0	0
3	84+00	4900	1282.5	Sandy Clay	3	39	58	40	19	2.66	19.8	100.0	0
5	86+00	5260	1280.7	Fat Clay	3	13	84			2.66	27.7	95.1	
7	77+80	4750	1265.5	Sandy Clay	8	26	66	45	24	2.72	20.4	101.0	0
8	73+82	4620	1255.3	Sandy Clay	2	32	66	43	22	2.64	19.8	104.0	0
9	86+75	4687	1302.9	Sandy Clay	7	27	65	43	20	2.69	16.6	98.0	0
10	78+42	4909	1271.4	Sandy Clay	0	35	65	47	25	2.68	21.0	101.0	0
11	74+00	4746	1264.8	Sandy Clay	7	27	65	44	22	2.74	19.3	99.0	0
12	76+35	4947	1266.1	Sandy Clay	1	31	68	49	27	2.69	20.4	103.0	0
13	81+86	4893	1297.1	Sandy Clay	8	26	66	48	26	2.70	19.1	104.0	0
14	30+16	5330	1289.5	Fat Clay	0	15	85	64	38	2.65	25.3	96.0	0
15	75+51	4828	1299.7	Sandy Clay	6	31	63	46	25	2.66	19.3	99.0	0
16	78+00	4950	1308.2	Gr. Sandy Clay	17	28	55	48	28	2.67	19.5	108.0	0
17	82+30	5068	1314.0	Gr. Clay	33	16	51	70	41	2.72	26.6	93.0	0
C. Earthwork Stage III:													
1	90+00	4802	1315.4	Sandy Clay	13	24	63	38	18	2.67	21.0	101.0	0
2	95+00	4890	1322.9	Sandy Clay	2	24	74	42	20	2.70	18.7	96.0	0
3	92+82	5151	1317.7	Fat Clay	0	20	80	64	35	2.69	37.5	82.0	1
4	101+00	4479	1331.6	Sandy Clay	7	38	55	37	16	2.73	19.5	98.2	0
5	82+01	4775	1324.8	Sandy Clay	4	29	67	42	19	2.76	18.1	98.1	
6	85+92	5012	1330.8	Sandy Clay	4	22	74	39	20	2.73	20.1	99.0	0
7	105+98	4962	1372.6	Sandy Clay	2	25	73	42	21	2.76	19.1	101.0	0
8	114+14	5000	1382.7	Sandy Clay	3	29	68	38	17	2.75	19.6	101.5	
9	110+00	5025	1389.0	Sandy Clay	6	26	68	40	20	2.78	20.0	103.2	
10	98+60	4891	1358.2	Sandy Clay	5	31	64	40	18	2.75	18.0	100.9	
11	94+00	4905	1352.6	Sandy Clay	6	32	62	42	23	2.77	16.8	107.0	0
12	85+02	4867	1351.4	Sandy Clay	6	27	67	44	23	2.76	20.3	97.9	0
13	77+28	4899	1343.9	Sandy Clay	7	27	66	41	20	2.74	20.1	91.5	
14	92+03	4909	1354.5	Sandy Clay	7	23	70	42	23	2.70	21.5	104.0	0
15	80+35	5043	1355.1	Sandy Clay	2	35	63	38	18	2.69	22.5	97.0	0
16	94+59	5109	1358.3	Sandy Clay	6	33	61	44	23	2.77	22.3	103.5	
17	85+69	5061	1361.4	Sandy Clay	13	35	52	38	19	2.73	18.9	107.5	
18	80+02	4896	1362.8	Sandy Clay	12	23	65	40	18	2.73	21.4	94.4	
19	86+18	5030	1377.0	Sandy Clay	4	23	73	39	18	2.80	17.1	93.9	
20	98+66	4979	1392.9	Sandy Clay	6	23	71	38	18	2.74	18.6	90.6	
21	93+00	5000	1394.0	Sandy Clay	2	23	75	40	19	2.75	19.6	94.6	
22	58+60	4425	1254.2	Silt	0	3	97	31	5	2.61	15.4	91.6	
23	52+65	5260	1301.5	Silt	0	6	94	30	5	2.68	15.9	92.5	0
24	53+43	5177	1303.1	Fat Clay	0	11	89	75	48	2.79	24.8	89.8	u
D. Earthwork Stage IV:													
1	66+80	4985	1289.3	Fat Clay				60	34	2.71	23.1	87.0	0
2	69+12	4952	1300.3	Lean Clay				36	18	2.68	15.8	106.0	
3	63+91	5024	1308.3	Lean Clay				31	13	2.66	17.0	109.0	0
4	64+95	5185	1290.0	Lean Clay				38	21	2.66	16.6	107.0	0
5	63+74	4968	1323.0	Lean Clay				37	19	2.73	18.7	109.0	
6	61+00	4815	1338.0	Lean Clay				45	23	2.69	20.6	100.0	0
7	60+85	4897	1359.5	Lean Clay				43	23	2.72	19.6	104.0	0
E. Earthwork Stage V:													
2	150+35	20+16	1332.7	Sandy Clay	3	35	62	37	21	2.70	15.4	99.0	0.
3	151+10	19+68	1360.0	Sandy Clay	5	33	62	32	16	2.71	16.8	106.0	
4	40+00	5025	1355.0	Sandy Clay	5	30	65	45	27				
5	54+82	4984	1368.0	Fat Clay				68	38				
6	40+00	5125	1364.0	Lean Clay				44	23	2.64	20.9	99.0	1
7	15+40	5060	1385.0	Sandy Clay	0	27	73	45	26	2.68	18.4	80.0	1

1

FORT RANDALL DAM, PERMANENT RECORD SAMPLES (ROLLED EMBANKMENT)

Soil Classification	Mechanical Analysis GR. %	SA. %	FI %	LL	PI	S. G.	M. C. %	Dry Density PCF	Void Ratio	Direct Shear COH TSF	Tan ϕ	Unc. Comp. TSF 1	2	3	Perm. Coeff. CM/SEC
andy Clay	0	28	72			2.69	17.9	96.9							
andy Clay	0	36	64			2.70	17.5	108.6							
andy Clay	0	33	67			2.69	16.2	108.4							
andy Clay	0	33	67			2.70	19.3	105.8							
andy Clay	0	32	68			2.74	19.0	104.1							
andy Clay	3	24	73			2.71	21.7	100.1		0.70	0.51				
andy Clay	20	22	58			2.71	19.9	104.3		0.72	0.62				
layey Sand	4	56	40			2.70	17.4	98.2		0.20	0.73				
layey Sand	23	33	44			2.66	20.1	101.3		0.48	0.53				
layey Sand	6	46	48			2.74	14.0	100.3		0.20	0.65				
andy Clay	7	27	68			2.74	19.9	97.4		0.18	0.55				
andy Clay	3	44	53			2.70	20.5	99.0		0.35	0.51				
andy Clay	3	27	70			2.69	20.6	103.0		0.55	0.51				
andy Clay	2	23	75			2.75	21.3	99.3		0.45	0.57				
andy Clay	0	32	68			2.71	20.2	100.0		0.45	0.45				
andy Clay	3	26	71			2.69	21.1	101.3		0.45	0.45				
andy Clay	3	22	75			2.70	20.8	100.3		0.42	0.50				
andy Clay	0	22	78			2.68	21.8	100.6		0.40	0.57				
andy Clay	7	18	75	47	26	2.67	19.8	93.0	0.802	0.30	0.54				
andy Gr. Clay	23	20	57	44	22	2.72	19.2	99.0	0.710	0.62	0.49				
andy Clay	3	39	58	40	19	2.66	19.8	100.0	0.669	0.38	0.65				
t Clay	3	13	84			2.66	27.7	95.1							
andy Clay	8	26	66	45	24	2.72	20.4	101.0	0.676	0.42	0.57				
andy Clay	2	32	66	43	22	2.64	19.8	104.0	0.585	0.48	0.46				
andy Clay	7	27	65	43	20	2.69	16.6	98.0	0.721	0.25	0.56				
andy Clay	0	35	65	47	25	2.68	21.0	101.0	0.661	0.20	0.52				
andy Clay	7	27	65	44	22	2.74	19.3	99.0	0.738	0.40	0.52				
andy Clay	[illegible]	31	68	49	27	2.69	20.4	103.0	0.640	0.50	0.45	2.56			
andy Clay	5	26	66	48	26	2.70	19.1	104.0	0.630	0.60	0.55	2.30	3.42		
t Clay	0	15	85	64	38	2.65	25.3	96.0	0.732	0.65	0.42				
andy Clay	6	31	63	46	25	2.66	19.3	99.0	0.684	0.45	0.46				
r. Sandy Clay	17	28	55	48	28	2.67	19.5	108.0	0.548	0.63	0.45	1.54	1.48	1.74	
r. Clay	33	16	51	70	41	2.72	26.6	93.0	0.825	0.43	0.39				
ndy Clay	3	24	63	38	18	2.67	21.0	101.0	0.658	0.23	0.50	0.83	0.87		
ndy Clay	2	24	74	42	20	2.70	18.7	96.0	0.764	0.22	0.50	1.58			8.5×10^{-9}
Clay	0	20	80	64	35	2.69	37.5	82.0	1.049	0.20	0.36	0.59	0.58		2.3×10^{-6}
ndy Clay	7	38	55	37	16	2.73	19.5	98.2	0.732			2.03			1.7×10^{-8}
ndy Clay	4	29	67	42	19	2.76	18.1	98.1				2.34	2.22	2.14	
dy Clay	4	22	74	39	20	2.73	20.1	99.0	0.724	0.02	0.60	1.53			
ndy Clay	2	25	73	42	21	2.76	19.1	101.0	0.705			2.94			3.9×10^{-8}
ndy Clay	3	29	68	38	17	2.75	19.6	101.5				2.20			
ndy Clay	6	26	68	40	20	2.78	20.0	103.2				2.12			
dy Clay	5	31	64	40	18	2.75	18.0	100.9				1.41			
dy Clay	6	32	62	42	23	2.77	16.8	107.0	0.621	0.60	0.50	1.77			
dy Clay	5	27	67	44	23	2.76	20.3	97.9	0.882			1.51			1.5×10^{-6}
dy Clay	7	27	66	41	20	2.74	20.1	91.5				1.49			
dy Clay	7	23	70	42	23	2.70	21.5	104.0	0.597	0.55	0.40	2.06			
dy Clay	2	35	63	38	18	2.69	22.5	97.0	0.739	0.23	0.47	1.14			
dy Clay	6	33	61	44	23	2.77	22.3	103.5				1.55			
dy Clay	13	35	52	38	19	2.73	18.9	107.5		0.39	0.43	2.08			
dy Clay	12	23	65	40	18	2.73	21.4	94.4				1.13			8.6×10^{-8}
dy Clay	4	23	73	39	18	2.80	17.1	93.9				1.58			
y Clay	6	23	71	38	18	2.74	18.6	90.6				0.68			
y Clay	2	23	75	40	19	2.75	19.6	94.6		0.10	0.52	1.51			
	0	3	97	31	5	2.61	15.4	91.6				0.91			1.3×10^{-5}
	0	6	94	30	5	2.68	15.9	92.5	0.811	0	0.61	1.24	1.68		1.8×10^{-6}
lay	0	11	89	75	48	2.79	24.8	89.8	0.831			2.42	2.02		3.1×10^{-8}
lay				60	34	2.71	23.1	87.0	0.958	0.08	0.62				5.9×10^{-7}
Clay				36	18	2.68	15.8	106.0				1.04			
Clay				31	13	2.66	17.0	109.0	0.547			1.24			1.1×10^{-6}
Clay				38	21	2.66	16.6	107.0	0.554	0.35	0.58	2.55	1.97		
Clay				37	19	2.73	18.7	109.0				1.48			
Clay				45	23	2.69	20.6	100.0	0.689	0.18	0.58	2.38			
lay				43	23	2.72	19.6	104.0	0.641	0.60	0.47	2.66			1.3×10^{-7}
Clay	3	35	62	37	21	2.70	15.4	99.0	0.708	0.20	0.61	1.92			
Clay	5	33	62	32	16	2.71	16.8	106.0				1.22			1.18×10^{-7}
Clay	5	30	65	45	27										
y				68	38										
lay				44	23	2.64	20.9	99.0	1.400	0.15	0.58	2.30	2.18		
Clay	0	27	73	45	26	2.68	18.4	80.0	1.089	0.10	0.51	1.28			2.8×10^{-7}

DATE	DESCRIPTION	MADE	APPR'D

REVISIONS

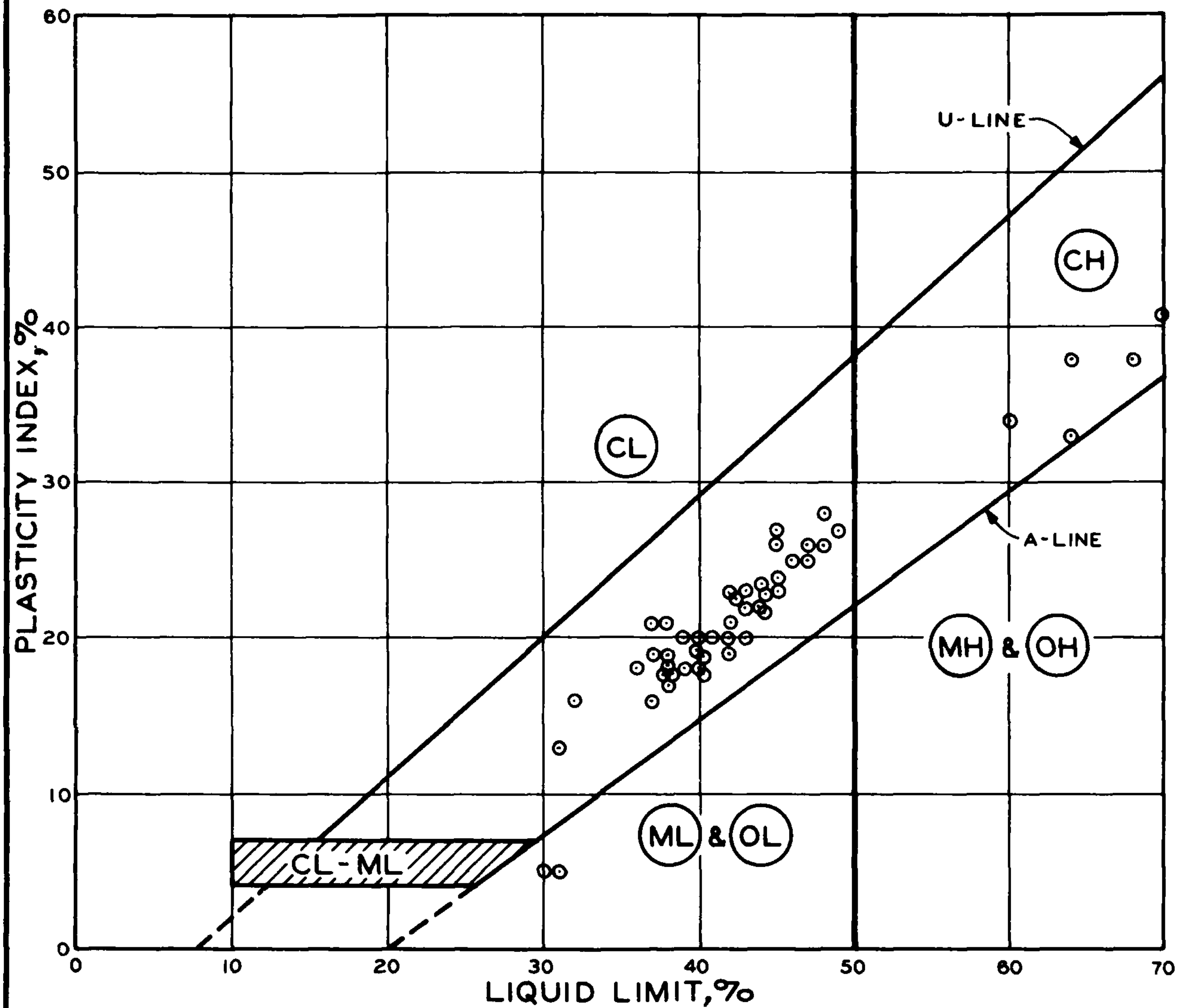
U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

MISSOURI RIVER
FORT RANDALL DAM-LAKE FRANCIS CASE

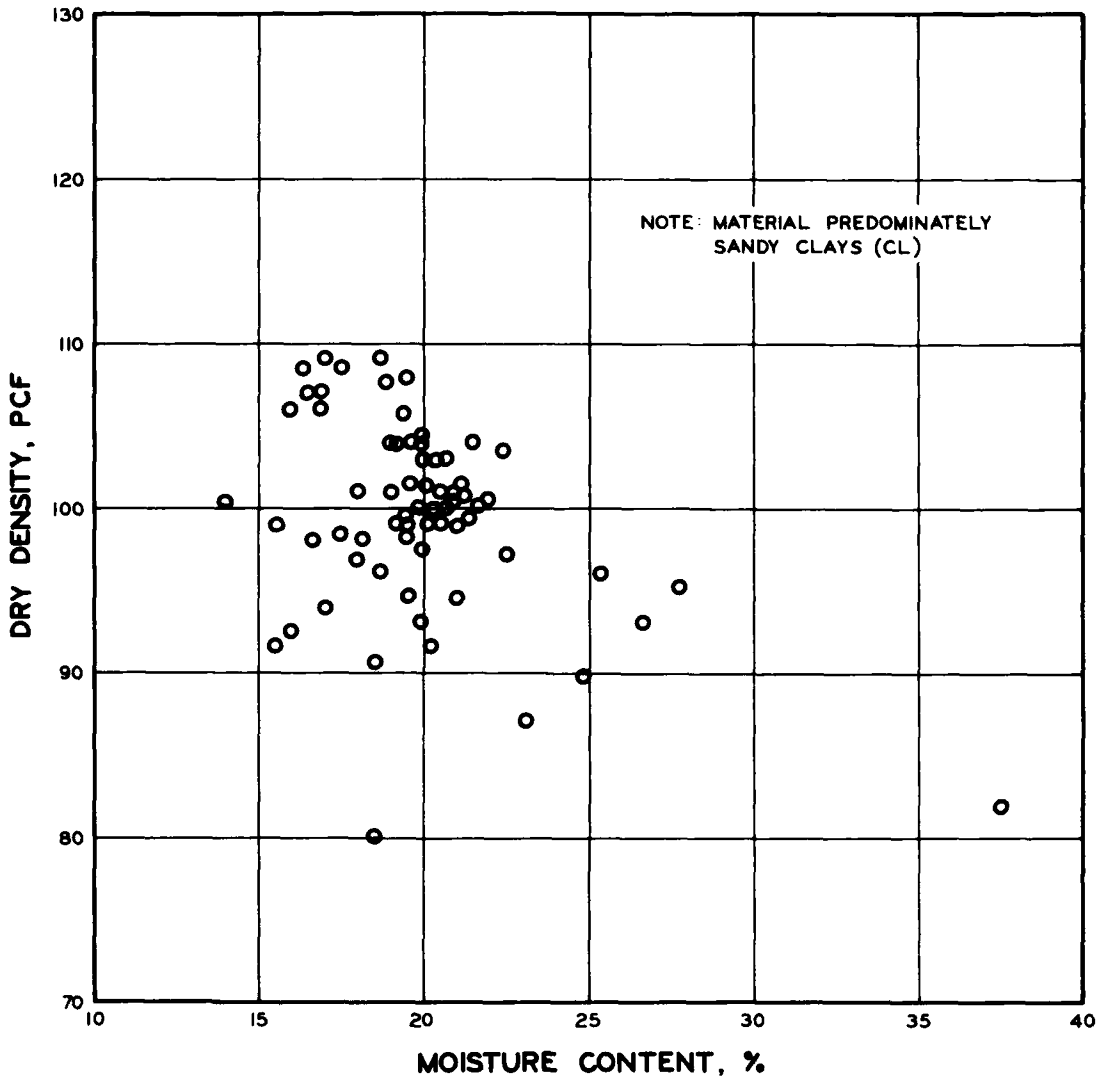
TABULATION OF TEST RESULTS ON PERMANENT RECORD SAMPLES

SCALE AS SHOWN

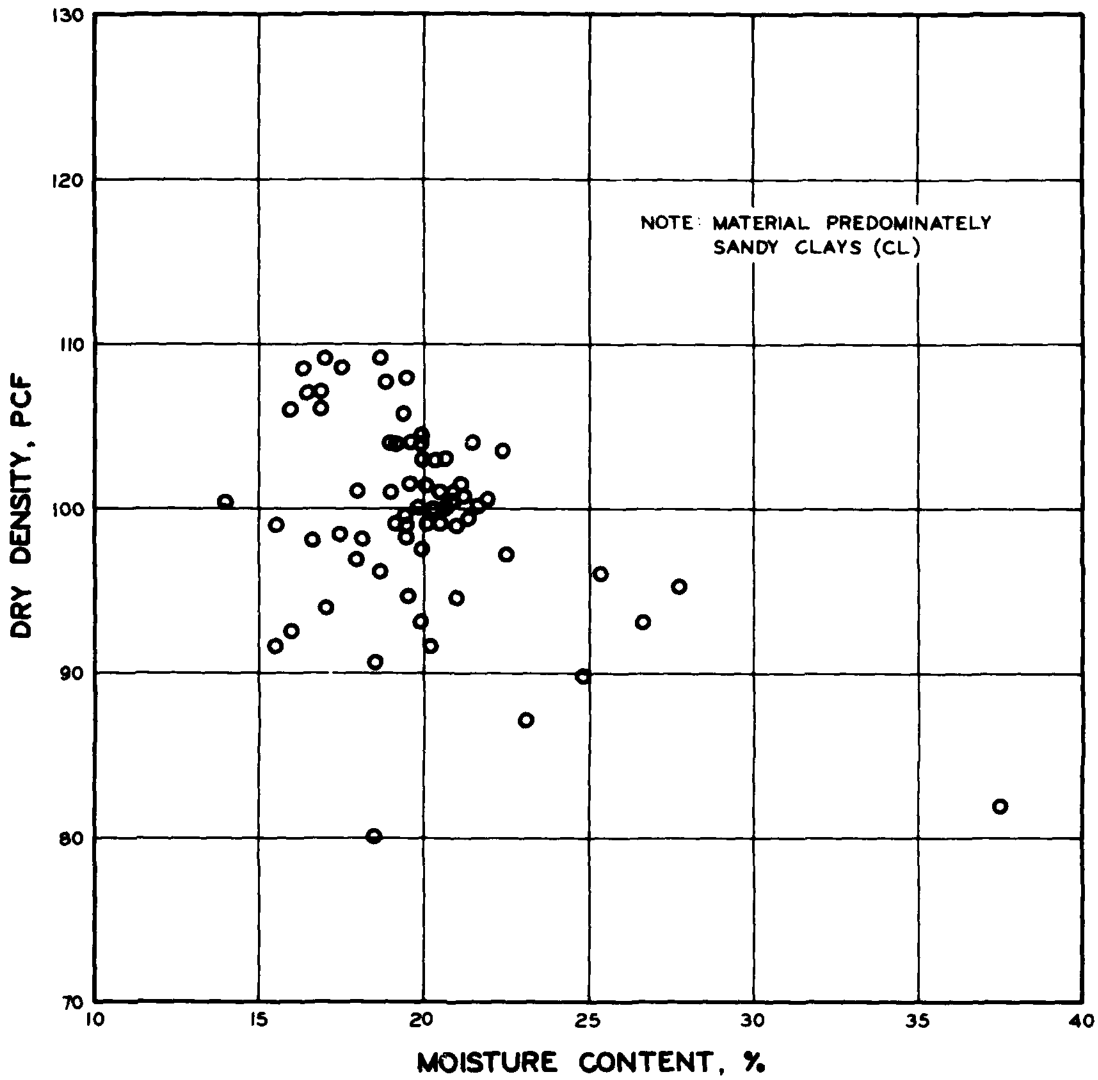
THIS PLAN ACCOMPANIES CONTRACT NO.
MODIFICATION NO.



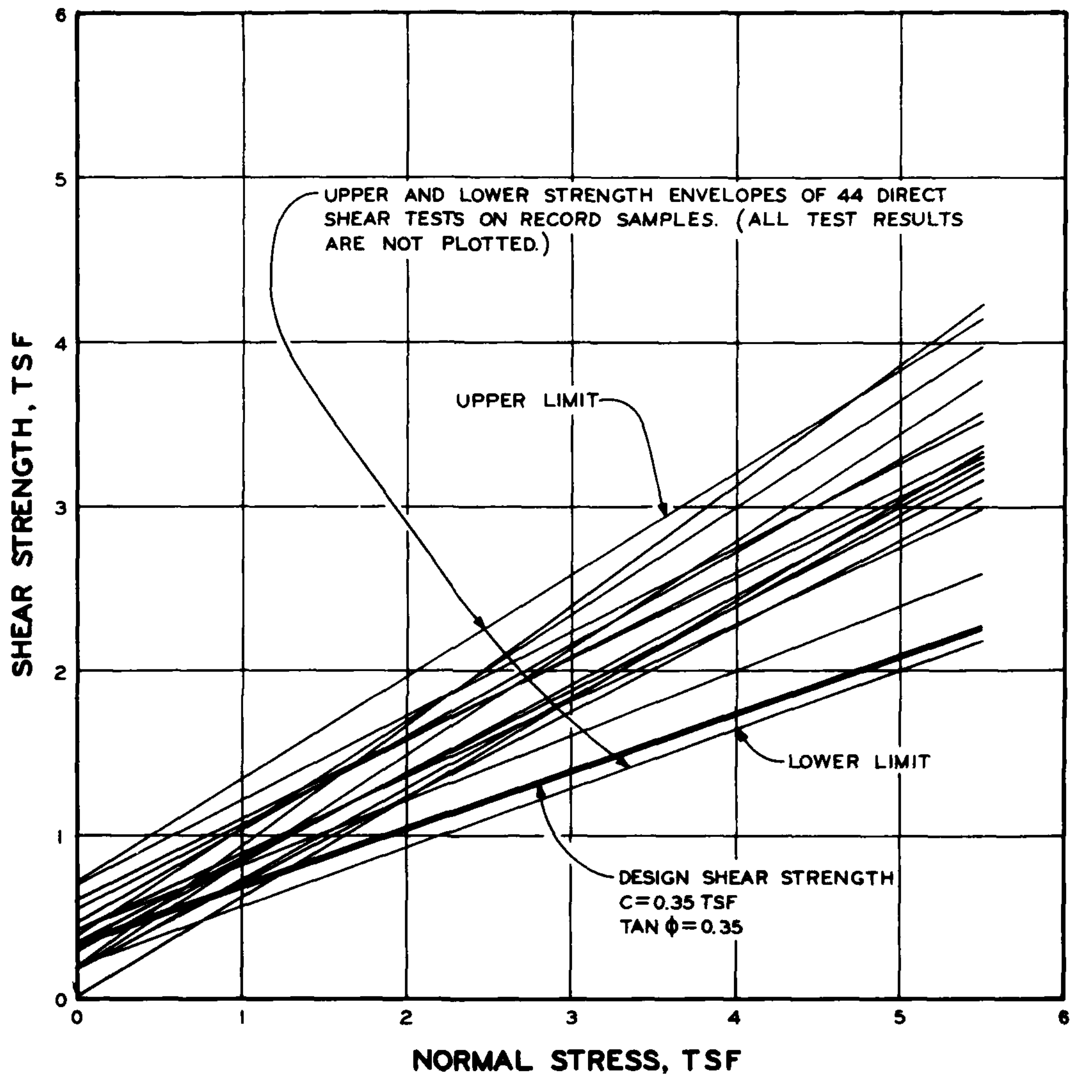
SUMMARY OF ATTERBERG LIMITS, EMBANKMENT RECORD SAMPLES FORT RANDALL DAM



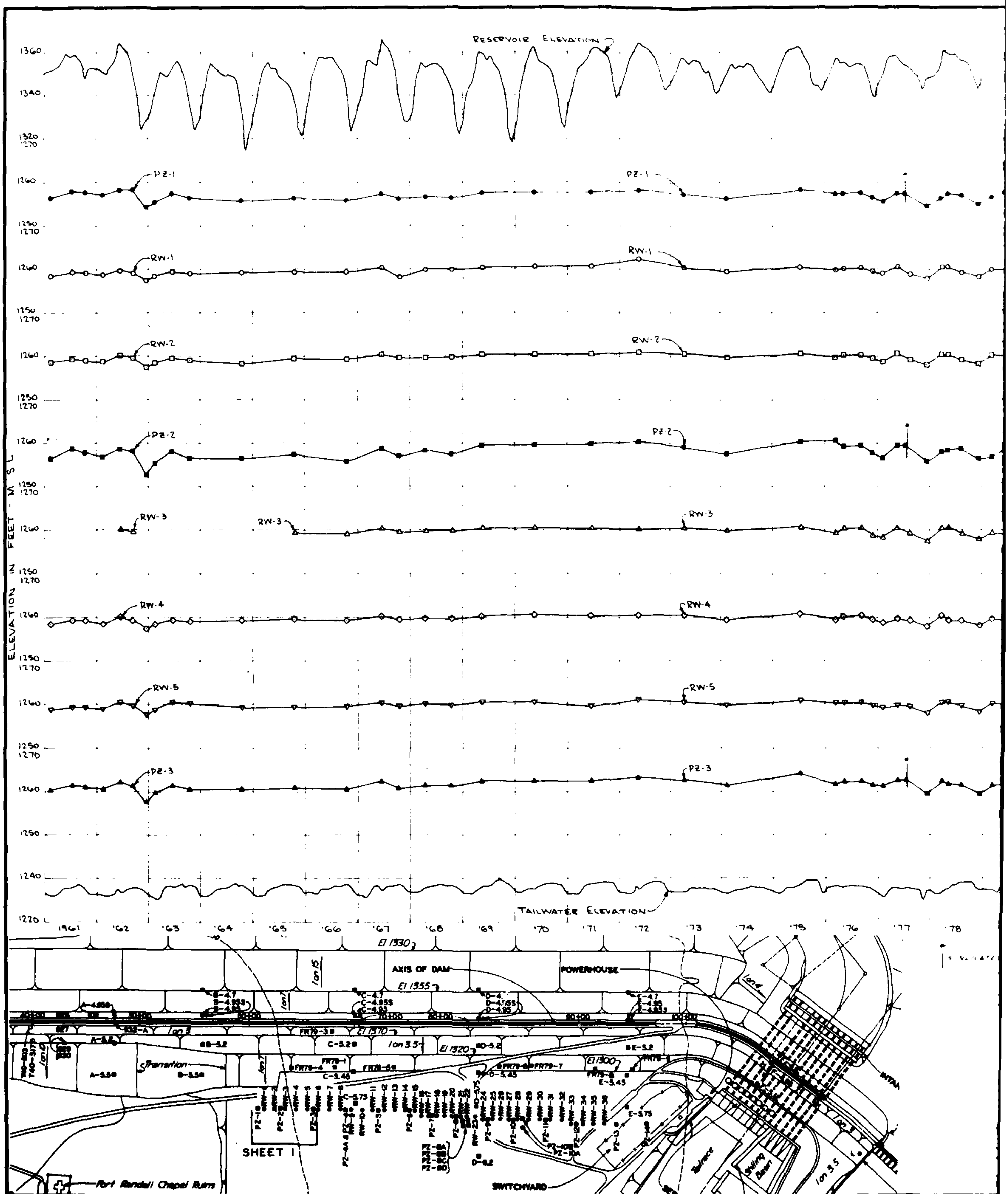
SUMMARY OF MOISTURE CONTENTS AND
DRY DENSITIES, EMBANKMENT RECORD SAMPLES
FORT RANDALL DAM



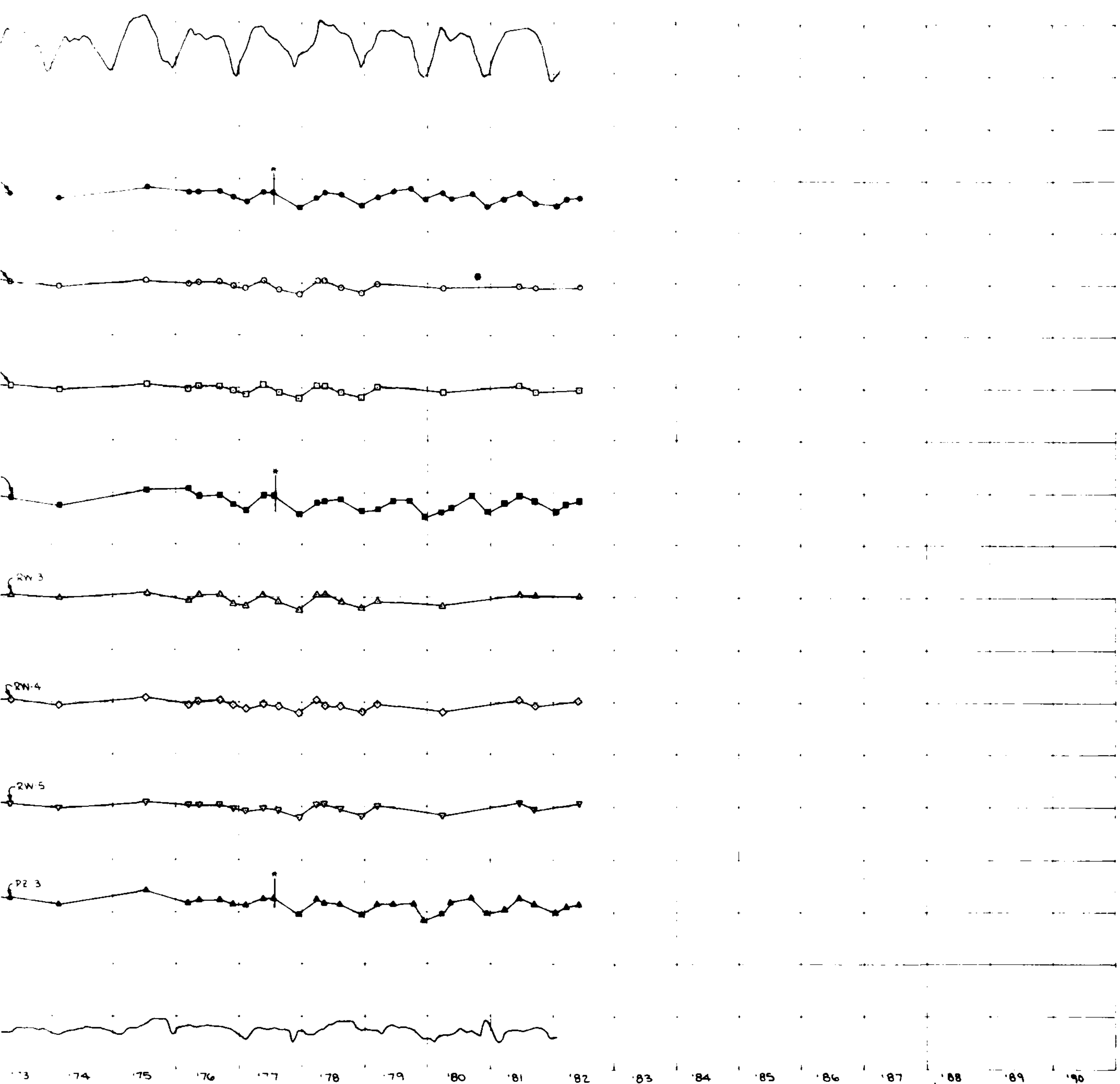
SUMMARY OF MOISTURE CONTENTS AND DRY DENSITIES, EMBANKMENT RECORD SAMPLES FORT RANDALL DAM

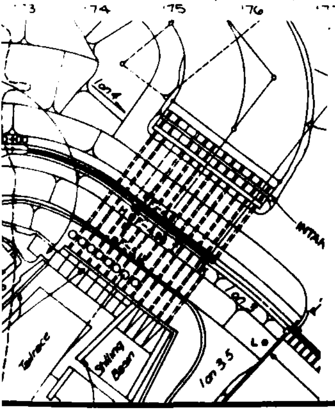


SUMMARY OF DIRECT SHEAR TESTS ON EMBANKMENT RECORD SAMPLES FORT RANDALL DAM



RESERVOIR ELEVATION
ELEVATION IN FEET - M S L
PZ-1
RW-1
RW-2
PZ-2
RW-3
RW-4
RW-5
PZ-3
TAILWATER ELEVATION
1961
'62
'63
'64
'65
'66
'67
'68
'69
'70
'71
'72
'73
'74
'75
'76
'77
'78
AXIS OF DAM
POWERHOUSE
SWITCHYARD
SHEET 1
Fort Randall Chapel Ruins
Transition
Tailrace
Stilling Basin





* = INDICATES PRESSURE EVALUATION TESTS

THIS DRAWING HAS BEEN REDUCED TO THREE-EIGHTHS THE ORIGINAL SCALE.

SAFETY PAYS

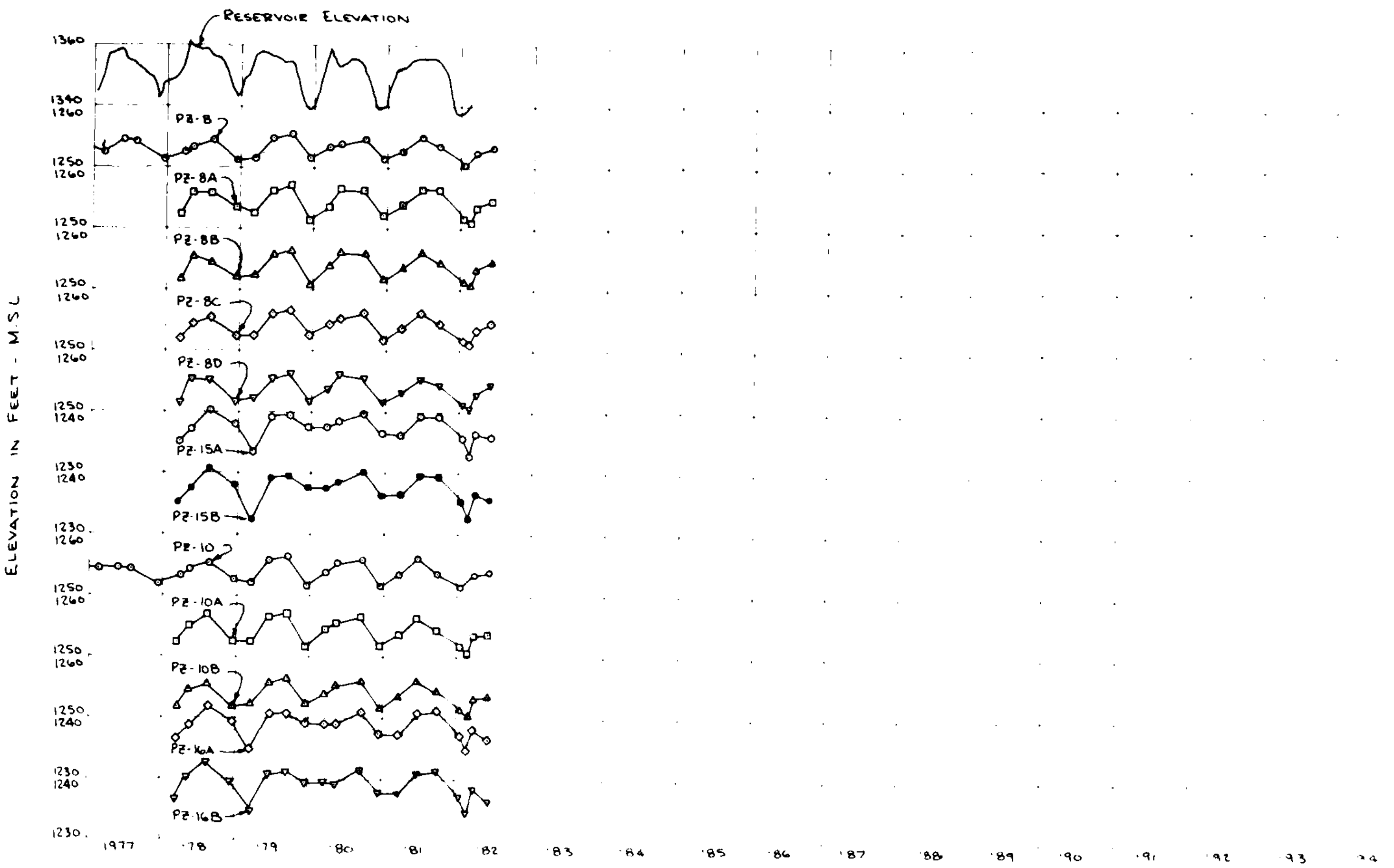
DATE	DESCRIPTION	MADE	APPR'D
	REVISIONS		

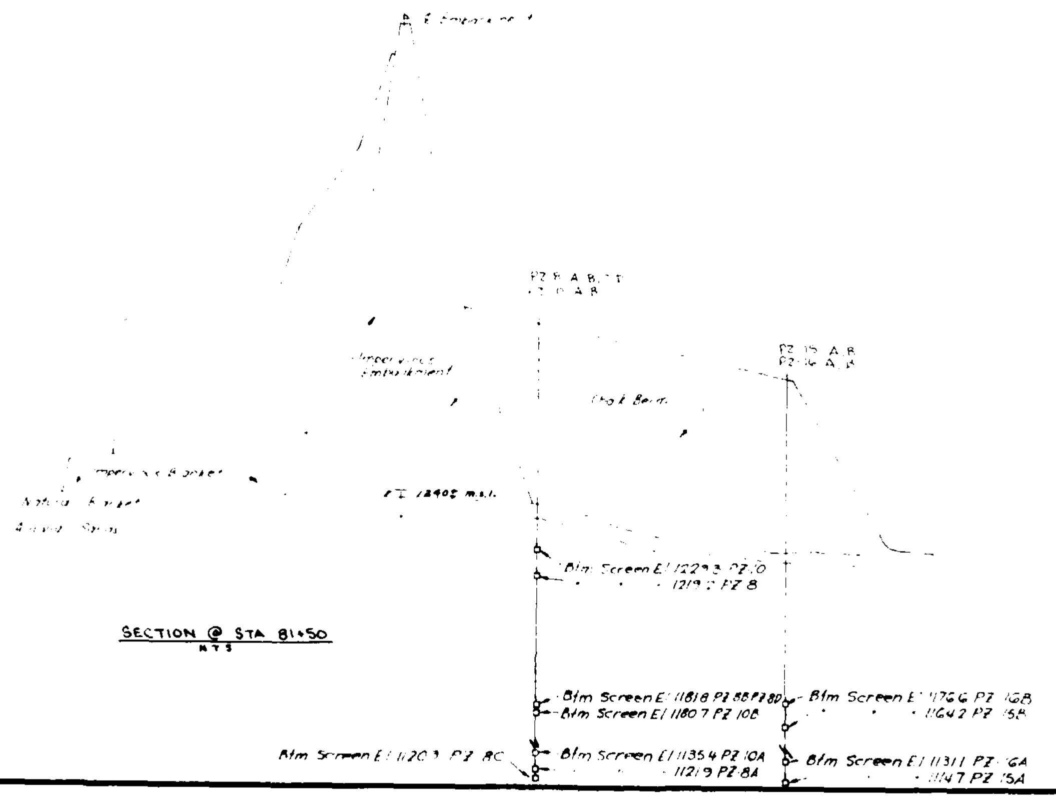
U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

DESIGNED BY:
DRAWN BY:
CHECKED BY:
SUBMITTED BY:
CHIEF ENG & D SECTION
RECOMMENDED:
CHIEF F & M BRANCH
APPROVED:
CHIEF ENGINEERING DIVISION
DATE:
SCALE: AS SHOWN
SPEC. NO.
DRAWING NUMBER
SHEET

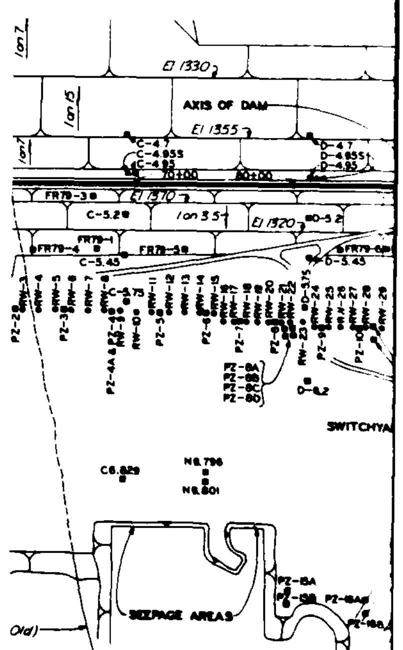
MISSOURI RIVER
FORT RANDALL DAM
EMBANKMENT
RELIEF WELL AND RELIEF WELL PIEZ.
WATER LEVEL OBSERVATIONS

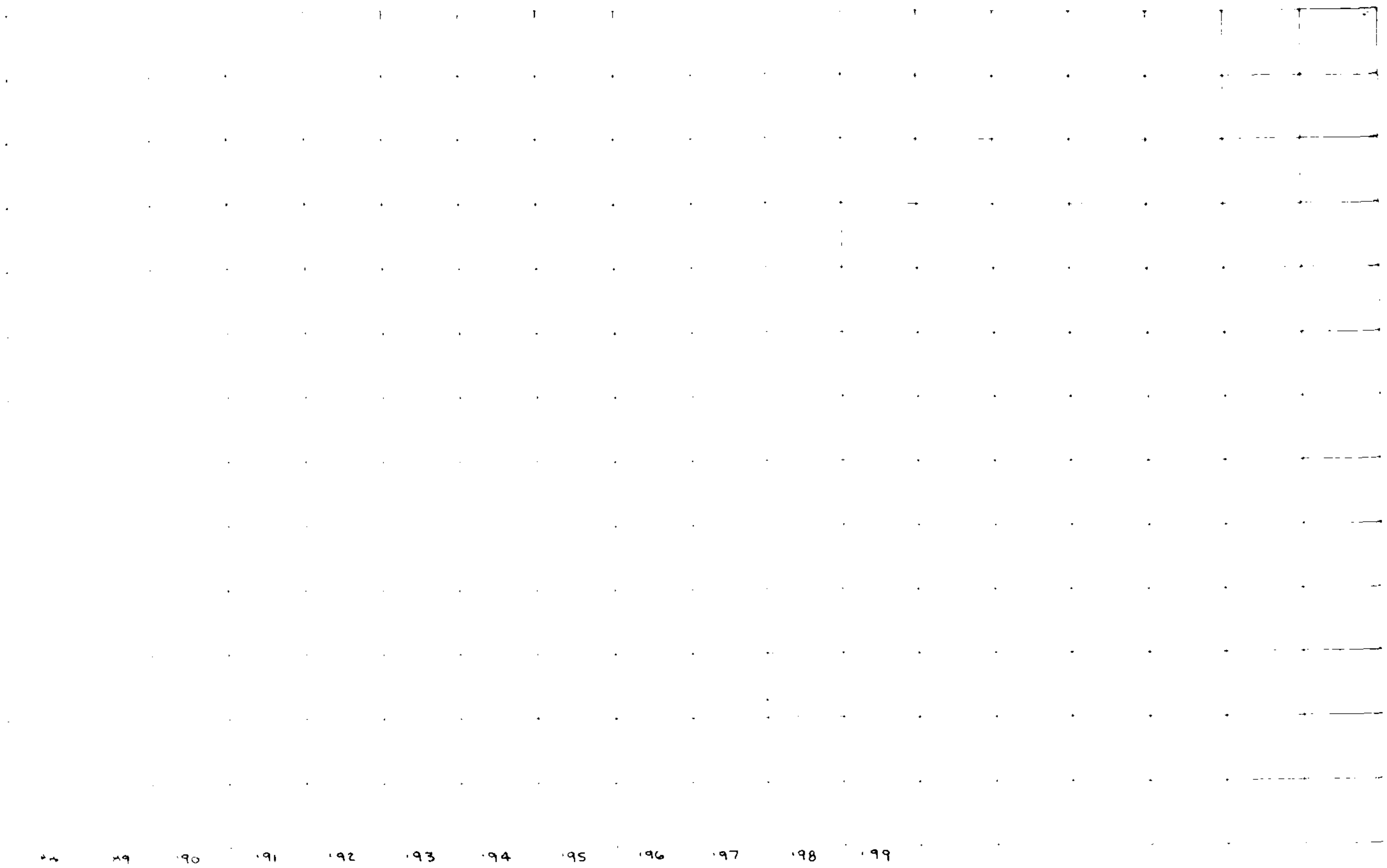
THIS PLAN ACCOMPANIES CONTRACT No.
MODIFICATION No.



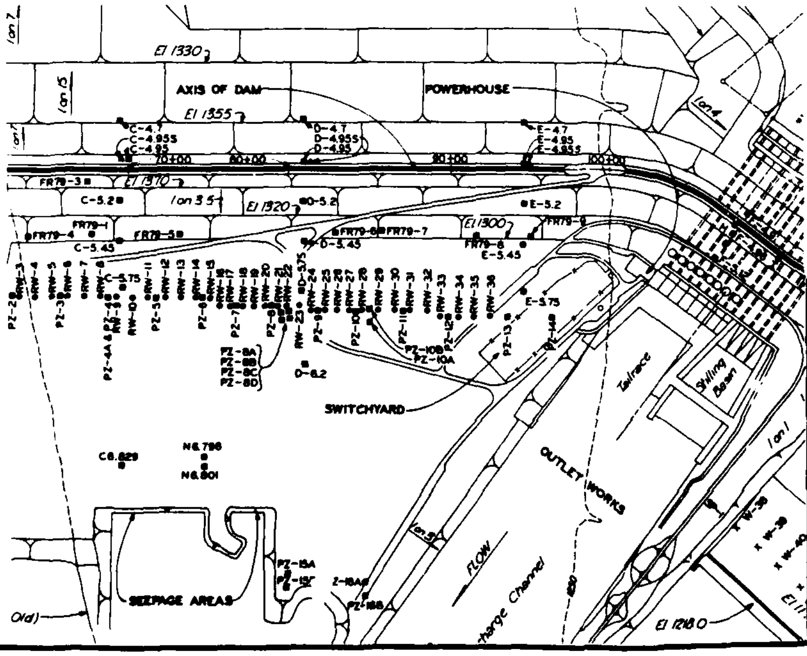


SECTION @ STA 81+50
N.T.S.





THIS DRAWING HAS BEEN REDUCED TO THREE-FOURTHS THE ORIGINAL SCALE.



DATE	DESCRIPTION	MADE	APPR'D

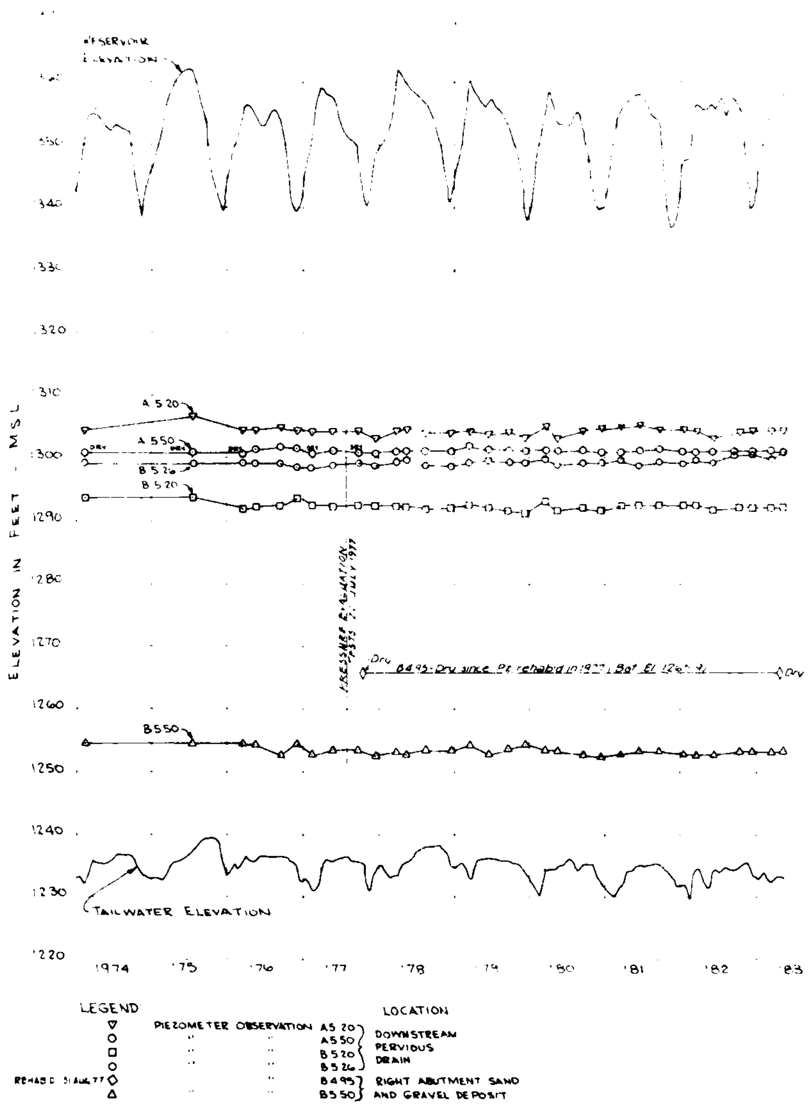
REVISIONS

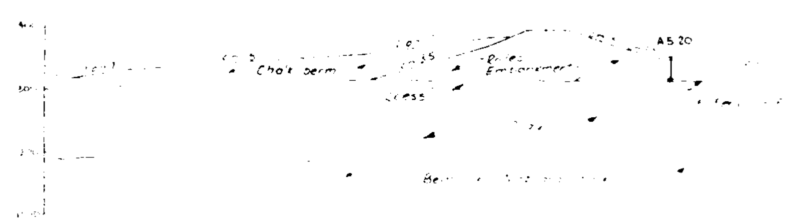
U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

MISSOURI RIVER
FORT RANDALL DAM
EMBANKMENT

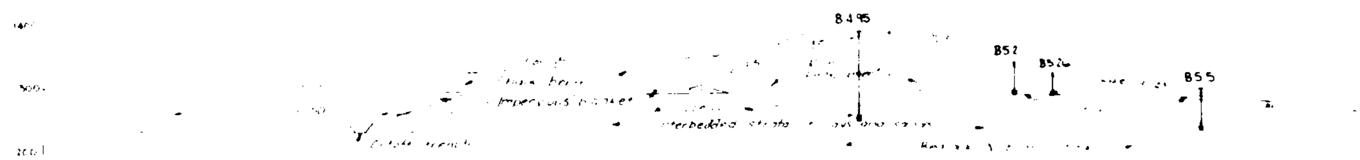
RELIEF WELL PIEZOMETERS PZ-8, 8A, 8B, 8C, 8D AND PZ-10, 10A, 10B & DOWNSTREAM VALLEY PZ'S 15A, 15B, 16A & 16B

SCALE AS SHOWN

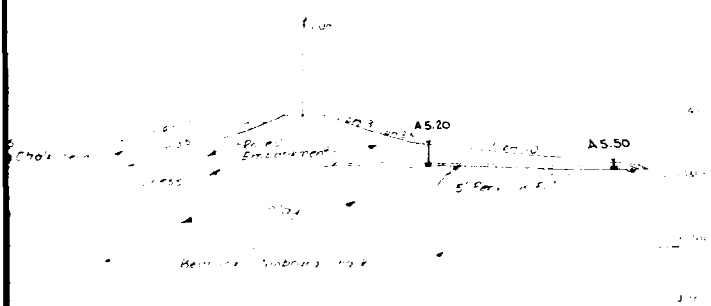




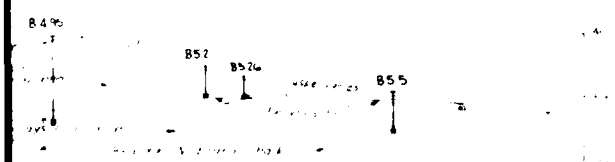
LINE A PIEZOMETERS



LINE B PIEZOMETERS



LINE "A" PIEZOMETERS



B PIEZOMETERS

THIS DRAWING HAS BEEN REDUCED TO THREE-EIGHTHS THE ORIGINAL SCALE

DATE	DESCRIPTION	MADE	APPR'D
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

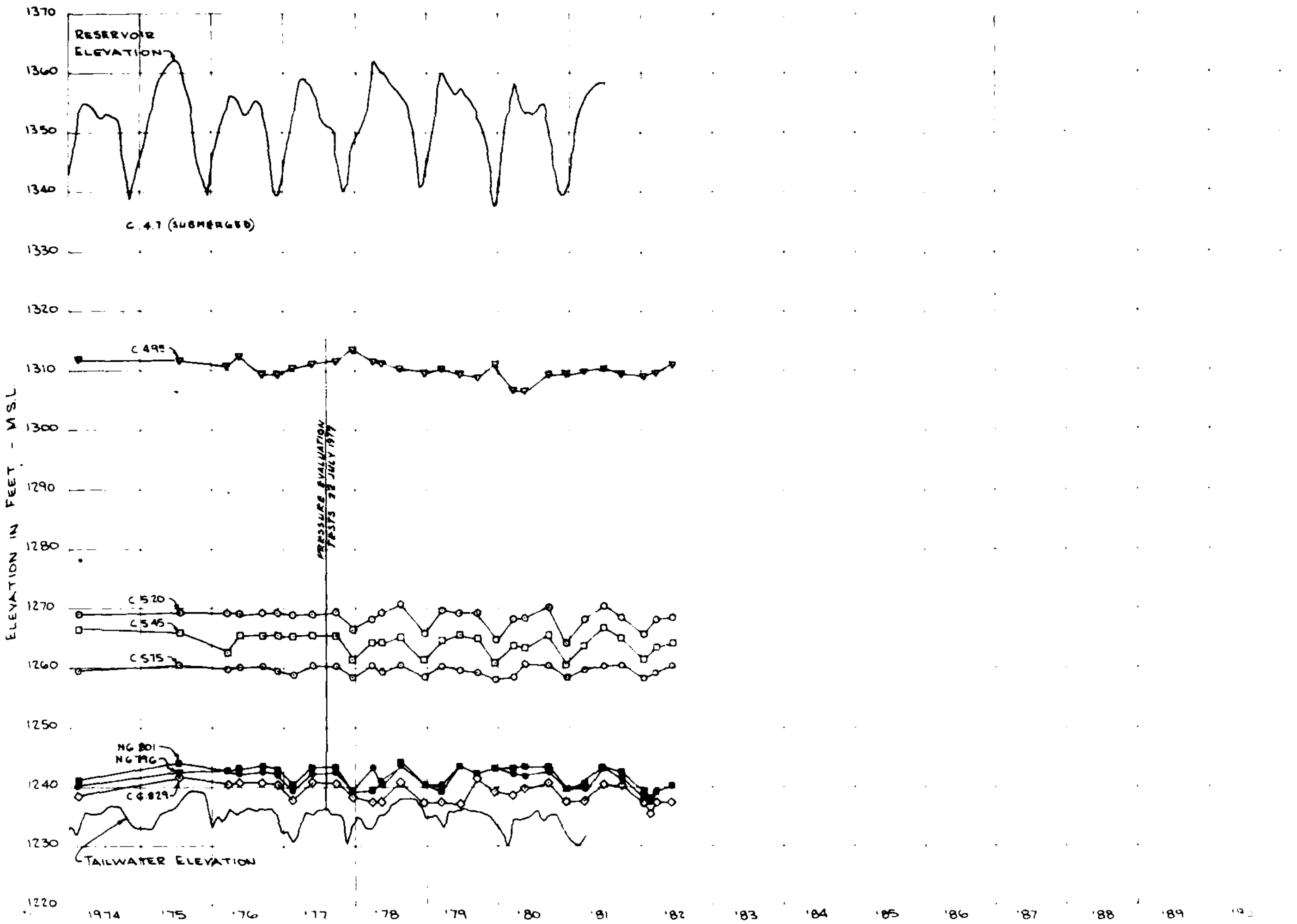
MISSOURI RIVER
FORT RANDALL DAM
EMBANKMENT
PIEZOMETER OBSERVATIONS
LINE A STA 48+00 - LINE B STA 56+00

DESIGNED BY: · DRAWN BY: · CHECKED BY: · SUBMITTED BY: · CHIEF SECTION · RECOMMENDED: · CHIEF BRANCH · APPROVED: · CHIEF ENGINEERING DIVISION · DATE · APPROVED: · COL. C. E., DISTRICT ENGINEER · SCALE: AS SHOWN · SPEC. NO. · DRAWING NUMBER · SHEET

SAFETY PAYS

THIS PLAN ACCOMPANIES CONTRACT No.
MODIFICATION No.

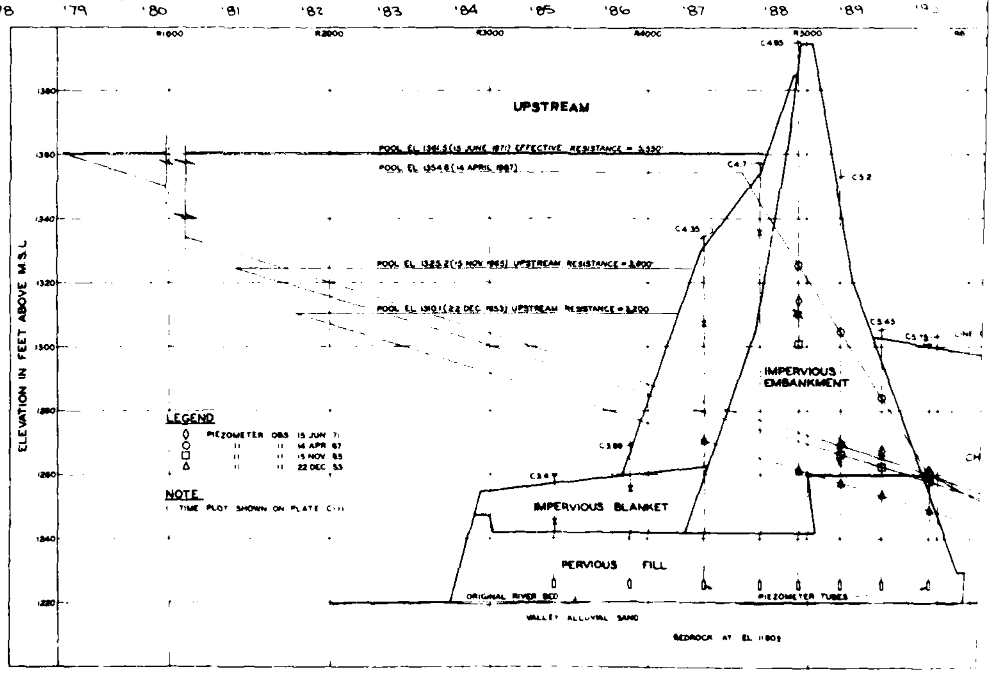
2

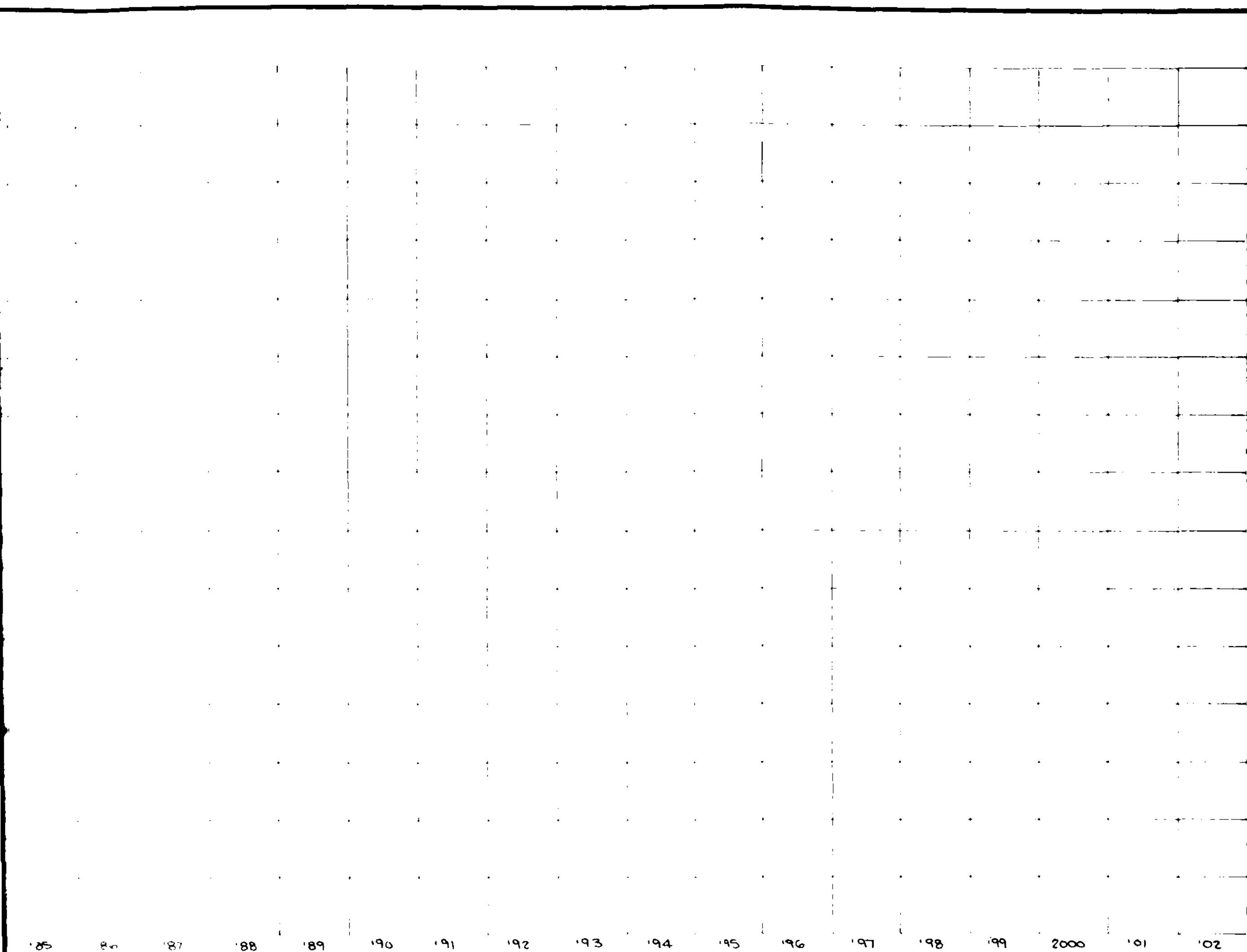


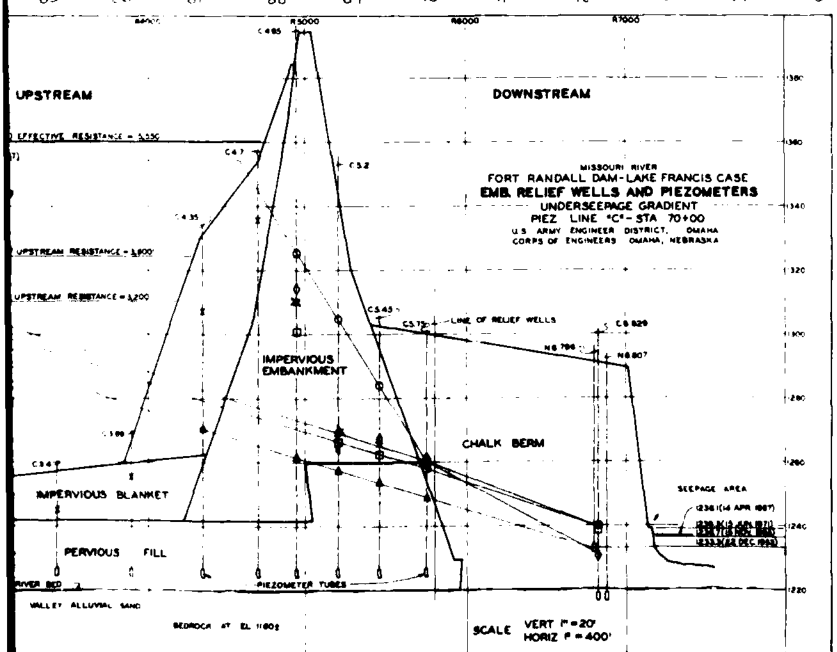
LEGEND

REHAB'D 29 AUG 77 ▽	PIEZOMETER OBSERVATION	C 495
REHAB'D 6 SEP 77 O	"	C 520
REHAB'D 31 AUG 77 □	"	C 545
O	"	C 575
◇	"	C G 829
●	"	N G 796
■	"	N G 801

Note: Piez. NG 796 and NG 801 are located at sta 75+31







THIS DRAWING HAS BEEN REDUCED TO THREE-EIGHTHS THE ORIGINAL SCALE

DATE	DESCRIPTION	MADE	APPR'D

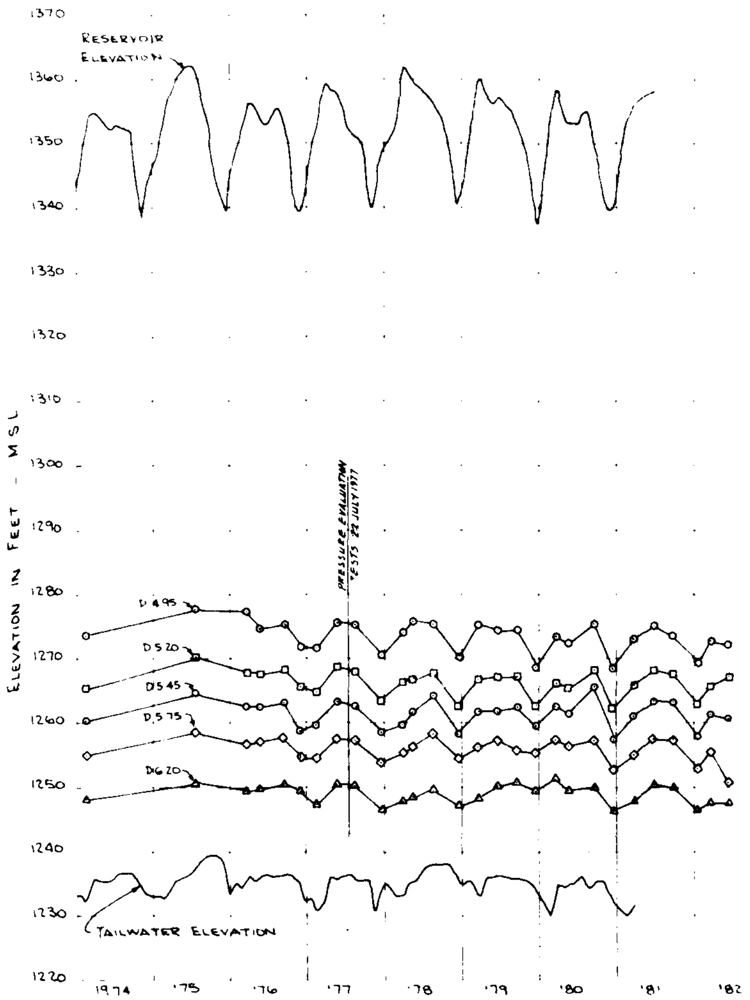
REVISIONS

U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

MISSOURI RIVER
FORT RANDALL DAM
EMBANKMENT
PIEZOMETER OBSERVATIONS
LINE C STA 70+00

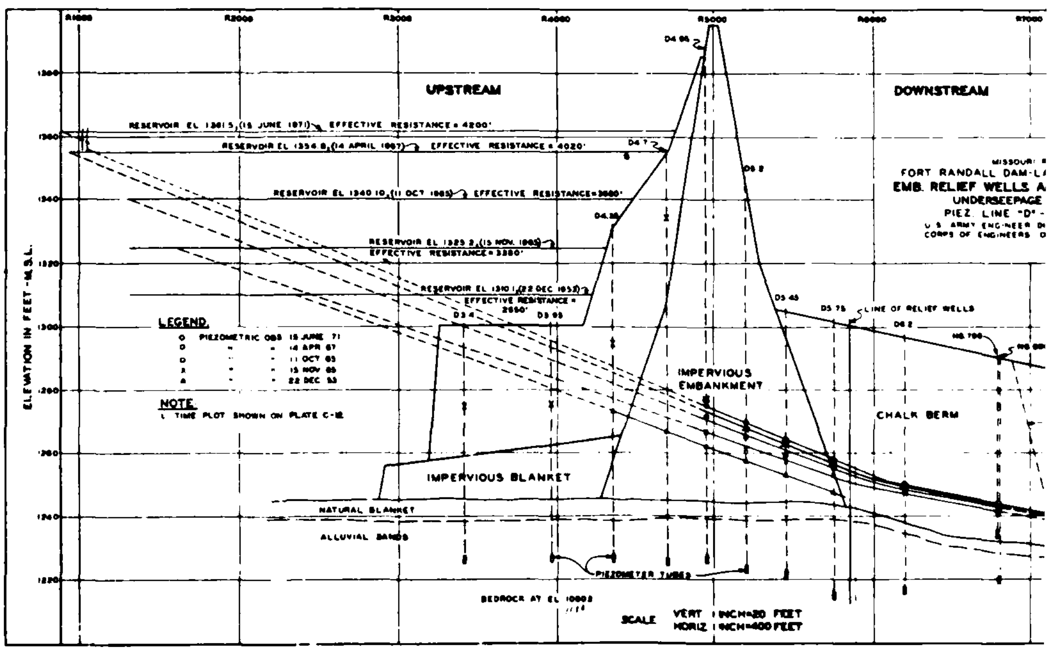
SCALE AS SHOWN

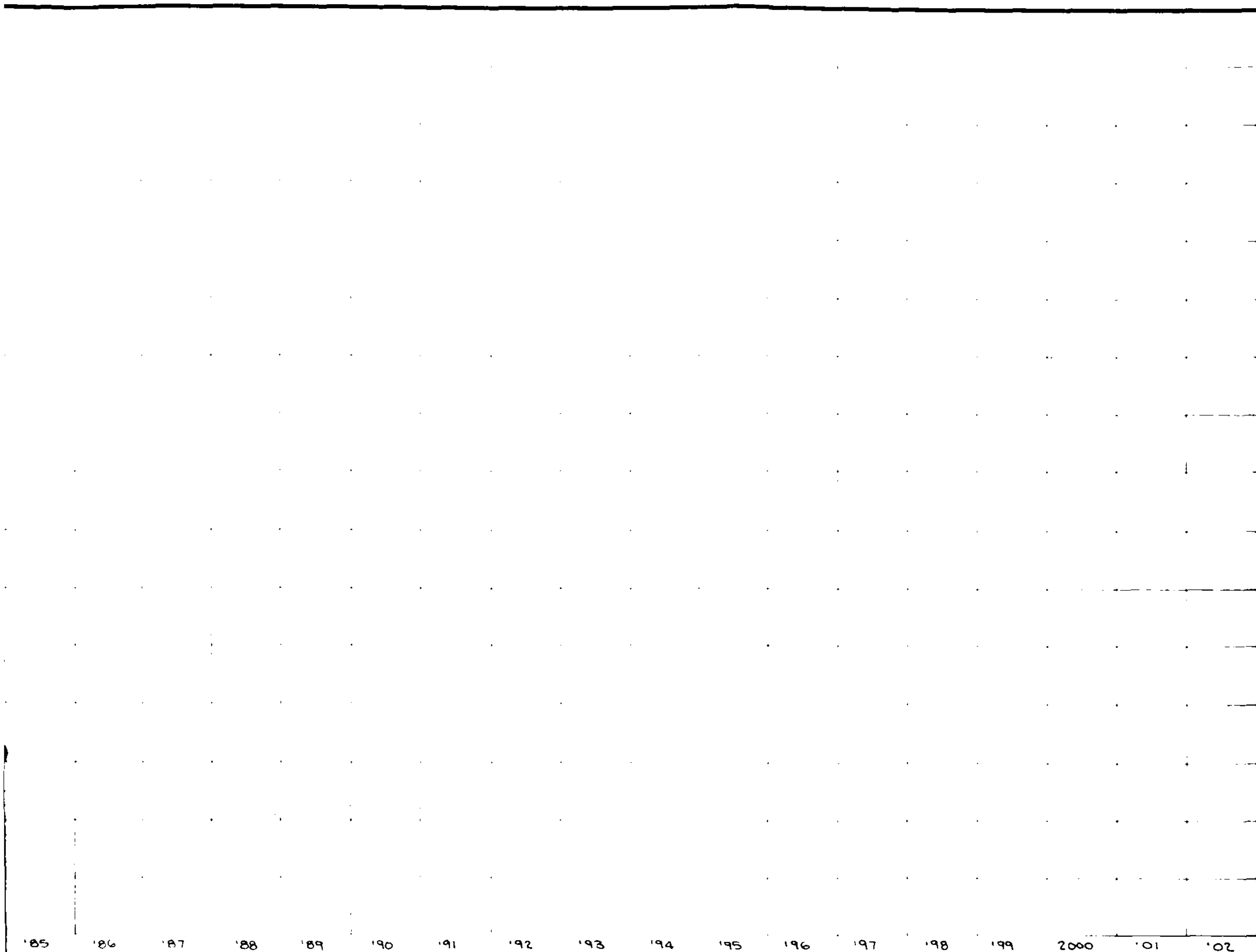
THIS PLAN ACCOMPANIES CONTRACT No.
MODIFICATION No.

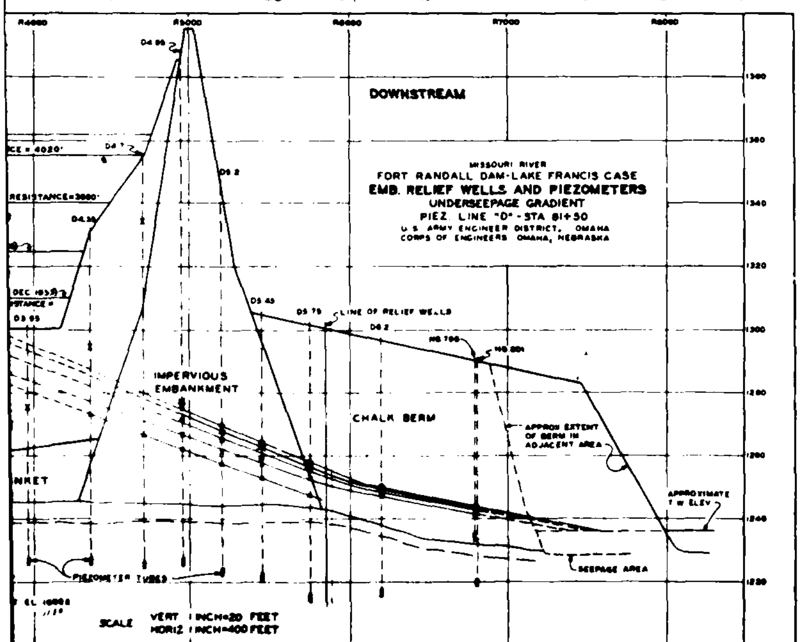


LEGEND
- ▽ PIEZOMETER OBSERVATION D4.70
- ○ D4.95
- □ D5.20
- ○ D5.45
- ◇ D5.75
- △ D6.20

x Piezometer inoperative







THIS DRAWING HAS BEEN REDUCED TO THREE-EIGHTHS THE ORIGINAL SCALE.

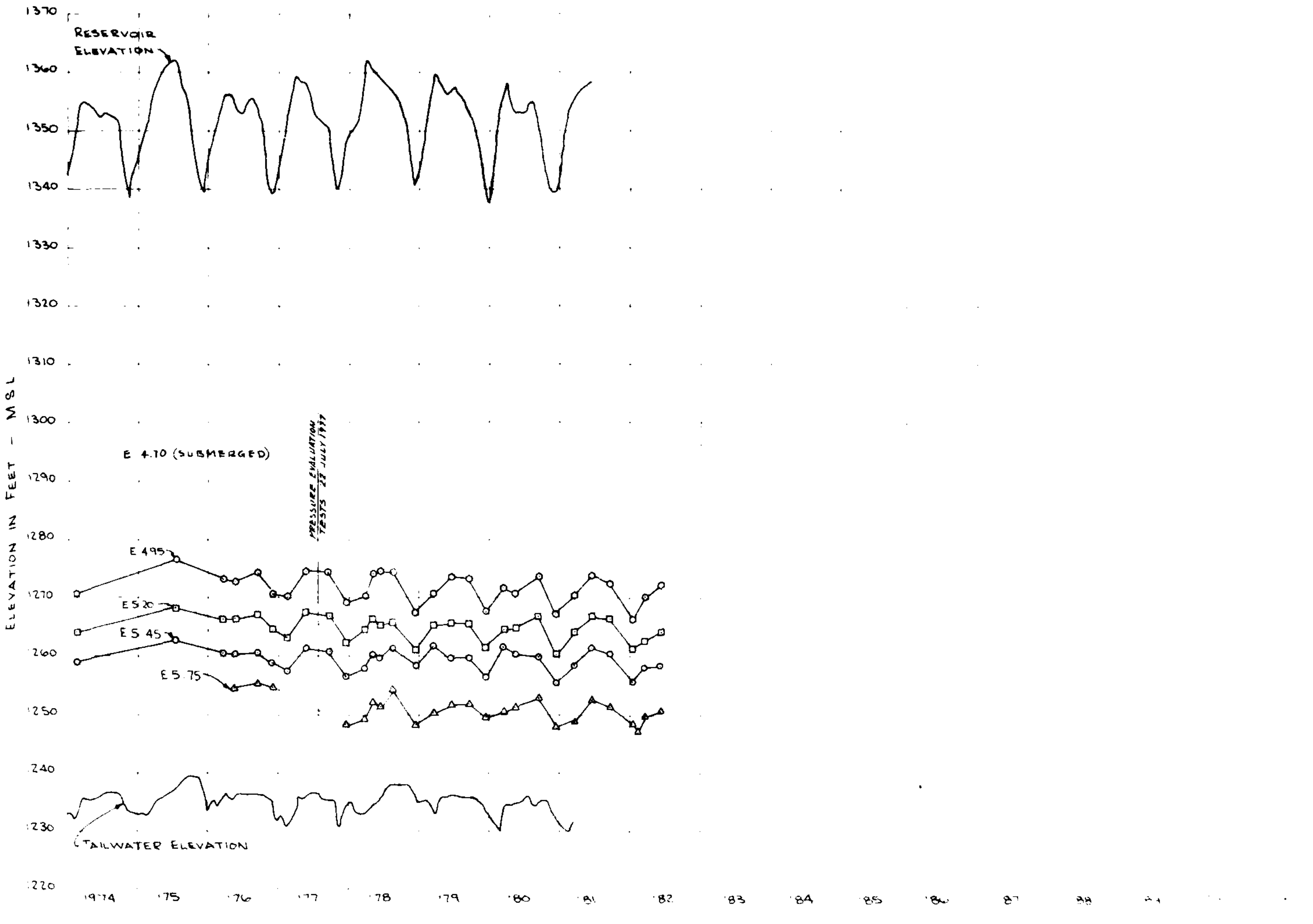
THIS PLAN ACCOMPANIES CONTRACT No.
MODIFICATION No.

DATE	DESCRIPTION	MAP.	APPR'D
REVISIONS			

U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

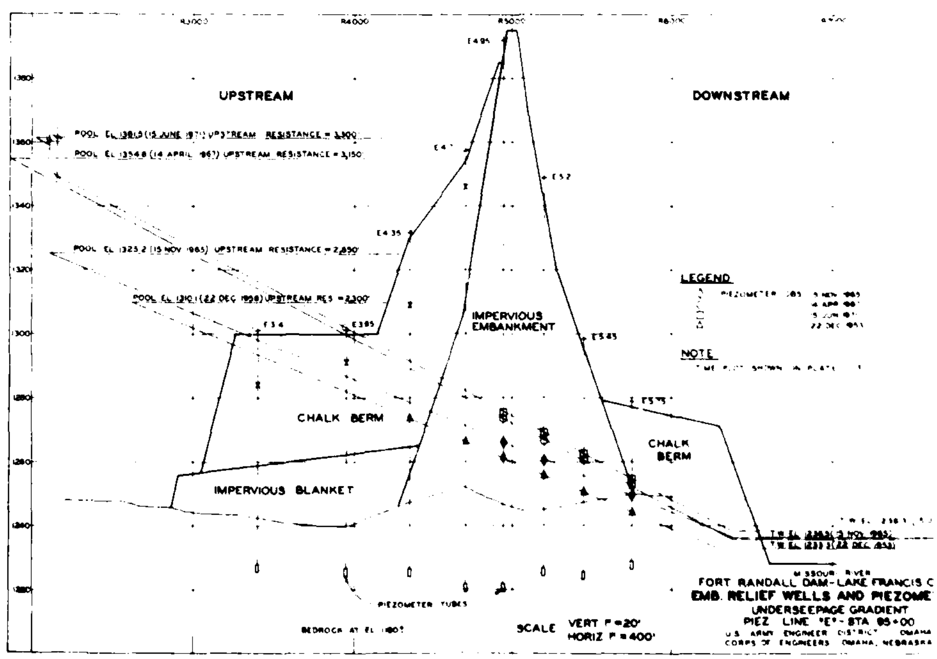
MISSOURI RIVER
FORT RANDALL DAM
EMBANKMENT
PIEZOMETER OBSERVATIONS
LINE D STA 81+60

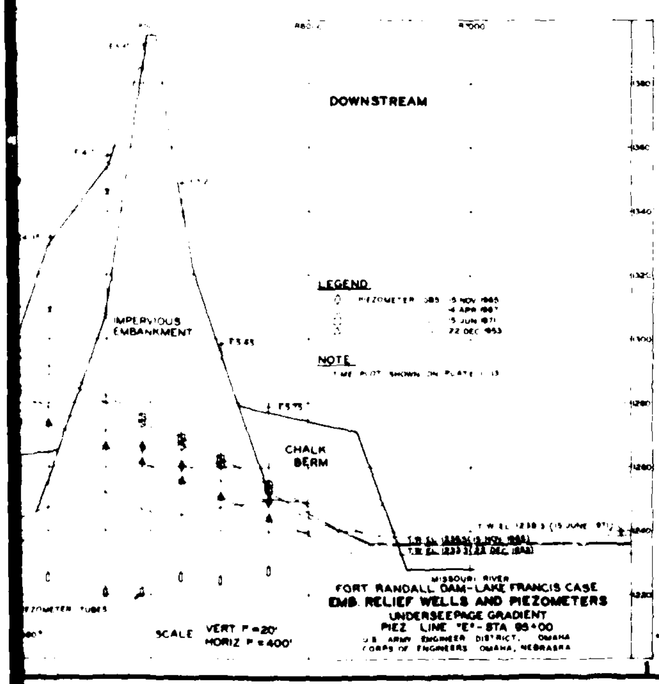
DESIGNED BY:
DRAWN BY:
CHECKED BY:
SUBMITTED BY:
APPROVED:
DATE:
SCALE AS SHOWN
DRAWING NUMBER
SHEET



LEGEND

▽	PIEZOMETER OBSERVATION	E4.70
O		E4.95
□		E5.20
O		E5.45
△		E5.75 REHAB'D 31 AUG 77





THIS DRAWING HAS BEEN REDUCED TO THREE-EIGHTHS THE ORIGINAL SCALE

DATE	DESCRIPTION	MARK	APPR'D

REVISIONS

U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

MISSOURI RIVER
FORT RANDALL DAM
EMBANKMENT

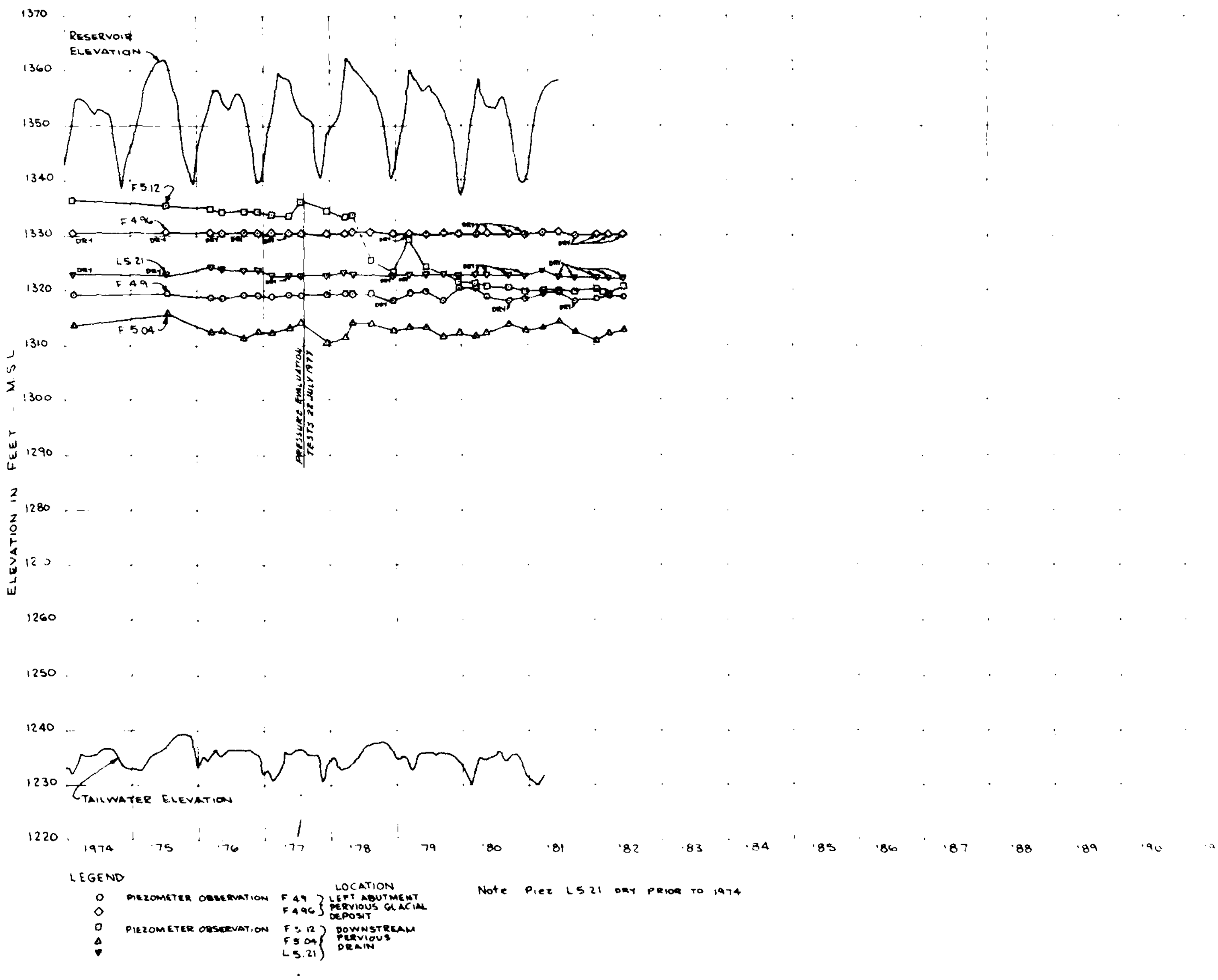
PIEZOMETER OBSERVATIONS
LINE E STA 95+00

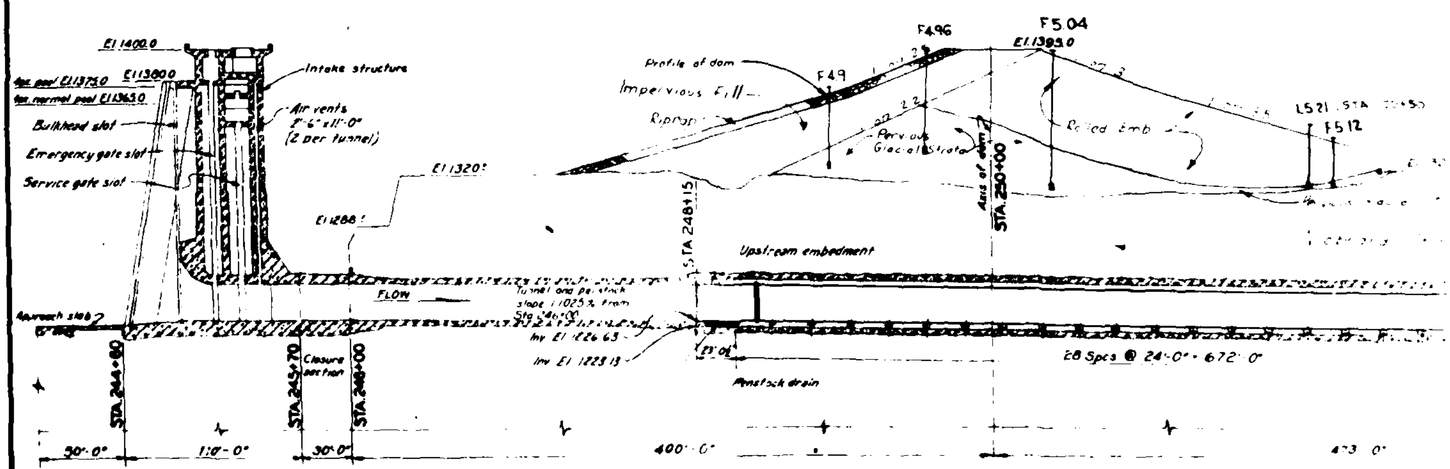
SCALE AS SHOWN

THIS PLAN ACCOMPANIES CONTRACT No.
MODIFICATION No.

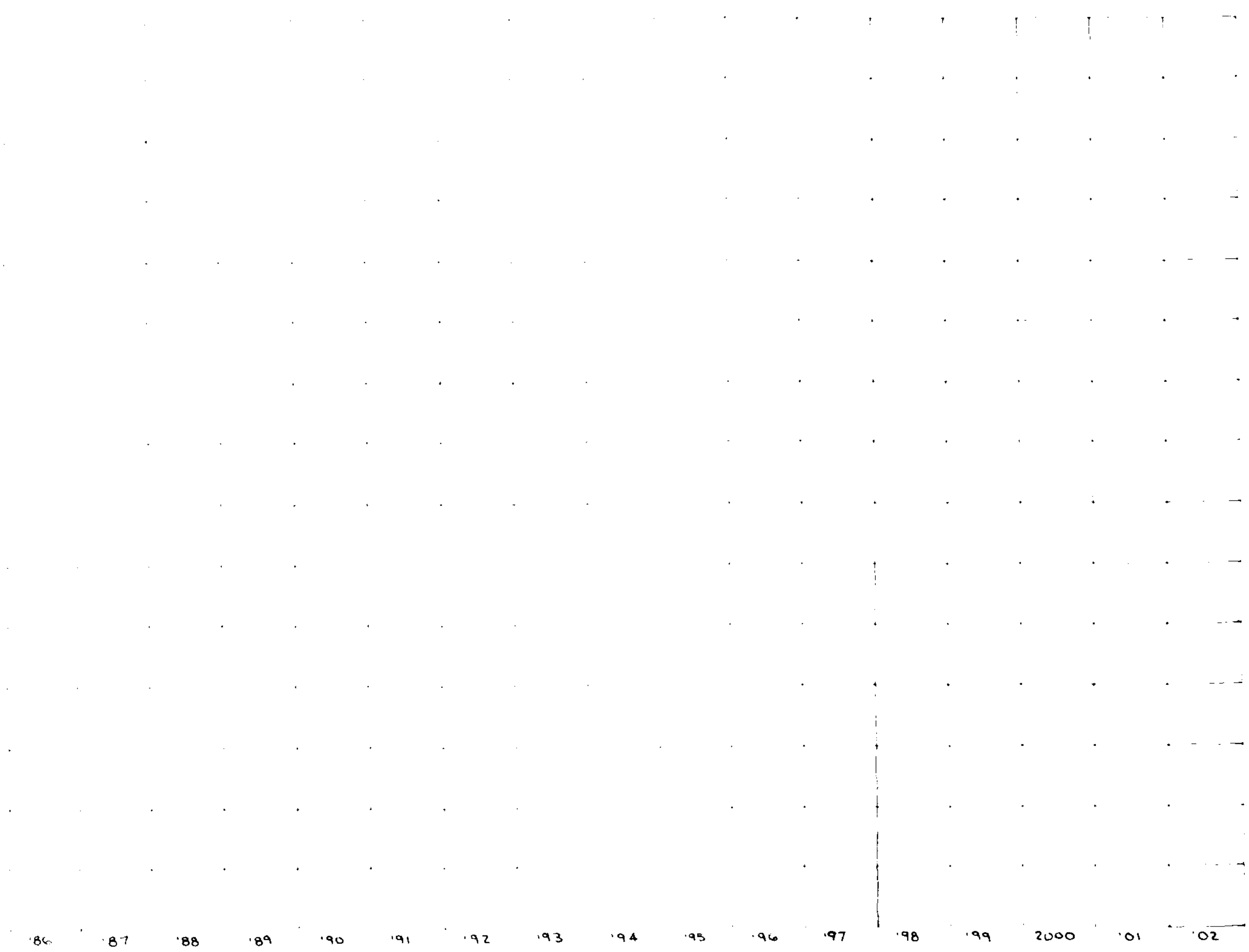
SAFETY PAYS

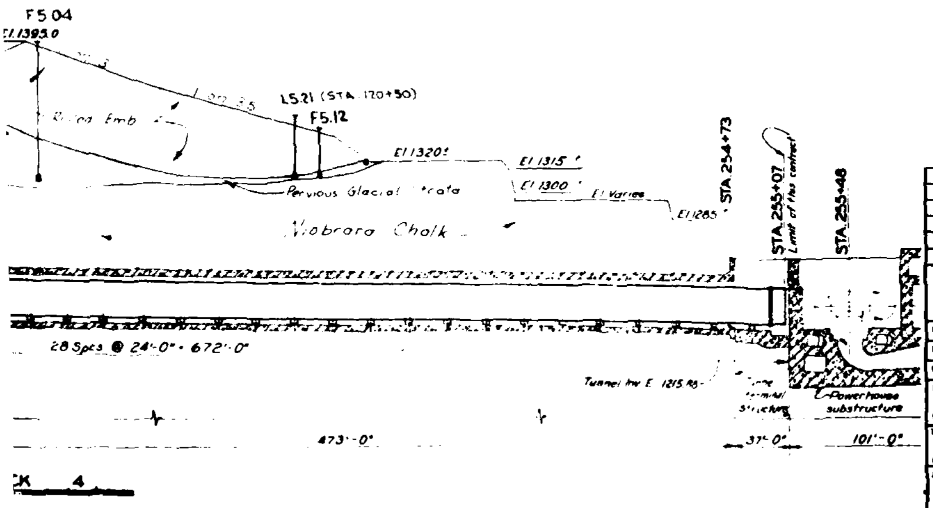
2





PROFILE OF PENSTOCK 4





THIS DRAWING HAS BEEN REDUCED TO THREE-EIGHTHS THE ORIGINAL SCALE.

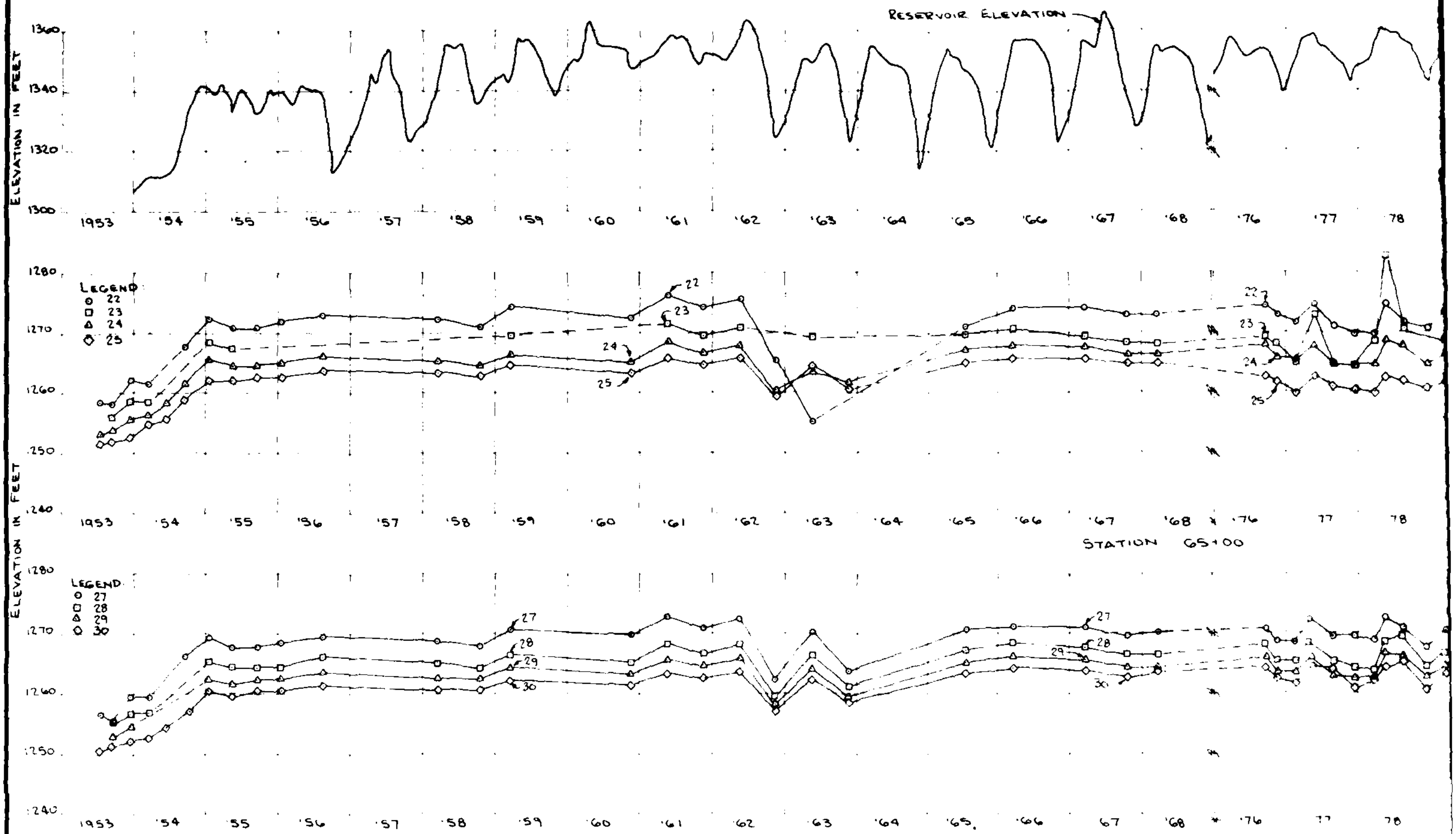
DATE	DESCRIPTION	MADE	APPR'D
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

MISSOURI RIVER
FORT RANDALL DAM
EMBANKMENT
LEFT ABUTMENT
PIEZOMETER OBSERVATIONS
LINE "F" 4 "L"

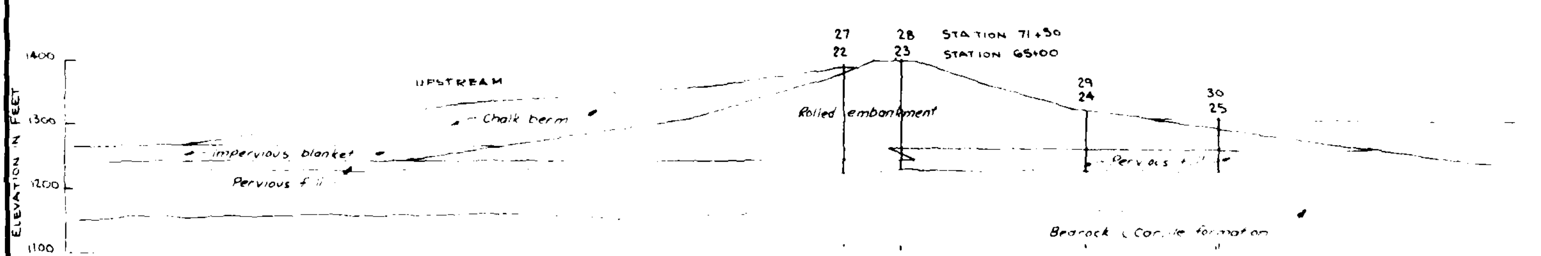
SCALE: AS SHOWN

2

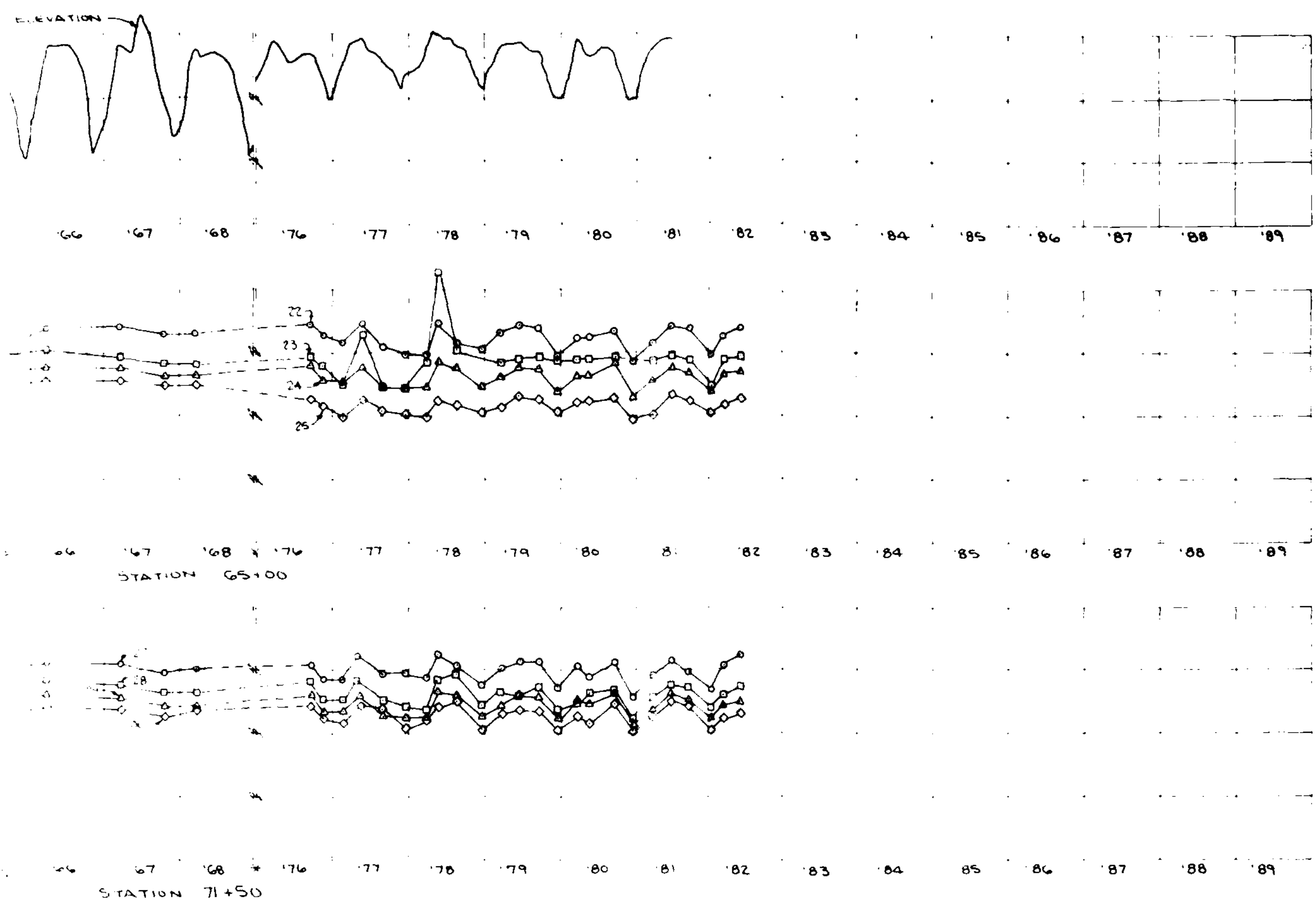


NOTE:

1. * No readings were recorded between 3-'68 & 9-'76
2. THESE [illegible] PIEZ'S WERE GIVEN PRESSURE EVALUATION TESTS 26 JUL & 18 AUG 1977



TYPICAL SECTION THRU EMBANKMENT CLOSURE SECTION

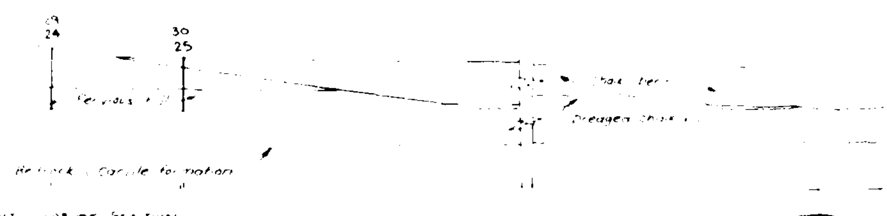


between 3-'68 to 9-'76

... PRESSURE EVALUATION TESTS 26 JUL & 18 AUG 1977

FOR LOCATION PLAN SEE PLATE NO. A53

THIS DRAWING HAS BEEN REDUCED TO THREE-EIGHTHS THE ORIGINAL SCALE.



DATE	DESCRIPTION	MADE	APPR'D

REVISIONS

U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

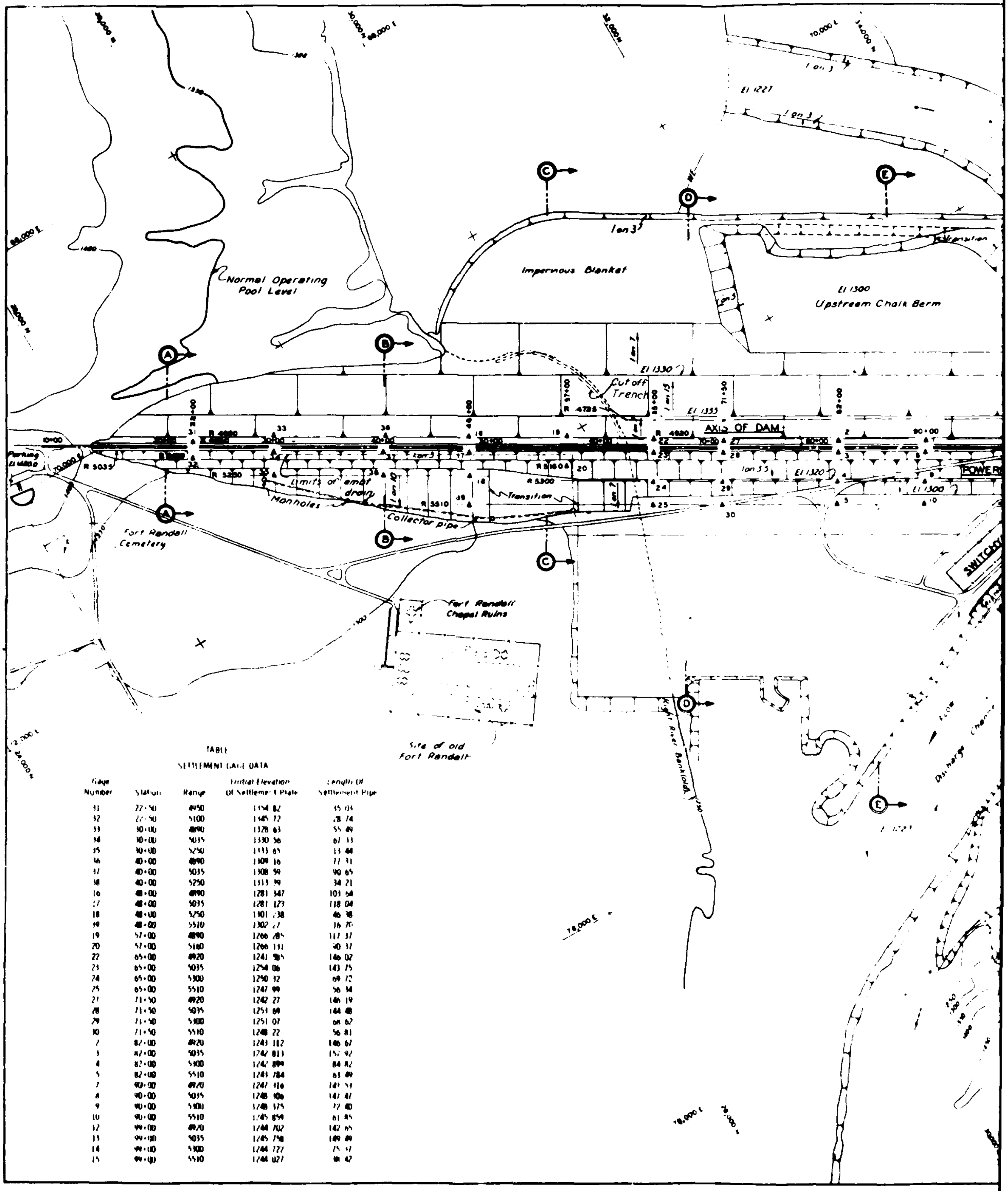
MISSOURI RIVER
FORT RANDALL DAM
SETTLEMENT GAGE PIEZOMETERS
STATIONS 65+00 AND 71+50
WATER SURFACE ELEVATIONS

DESIGNED BY:
DRAWN BY:
CHECKED BY:
SUBMITTED BY:
CHIEF SECTION
RECOMMENDED:
CHIEF BRANCH
APPROVED:
APPROVED: CHIEF ENGINEERING DIVISION
DATE:
SCALE AS SHOWN
SPEC. NO.
DRAWING NUMBER
COL. C. E. DISTRICT ENGINEER
SHEET 4

SAFETY PAYS

THIS PLAN ACCOMPANIES CONTRACT NO.
MODIFICATION NO.

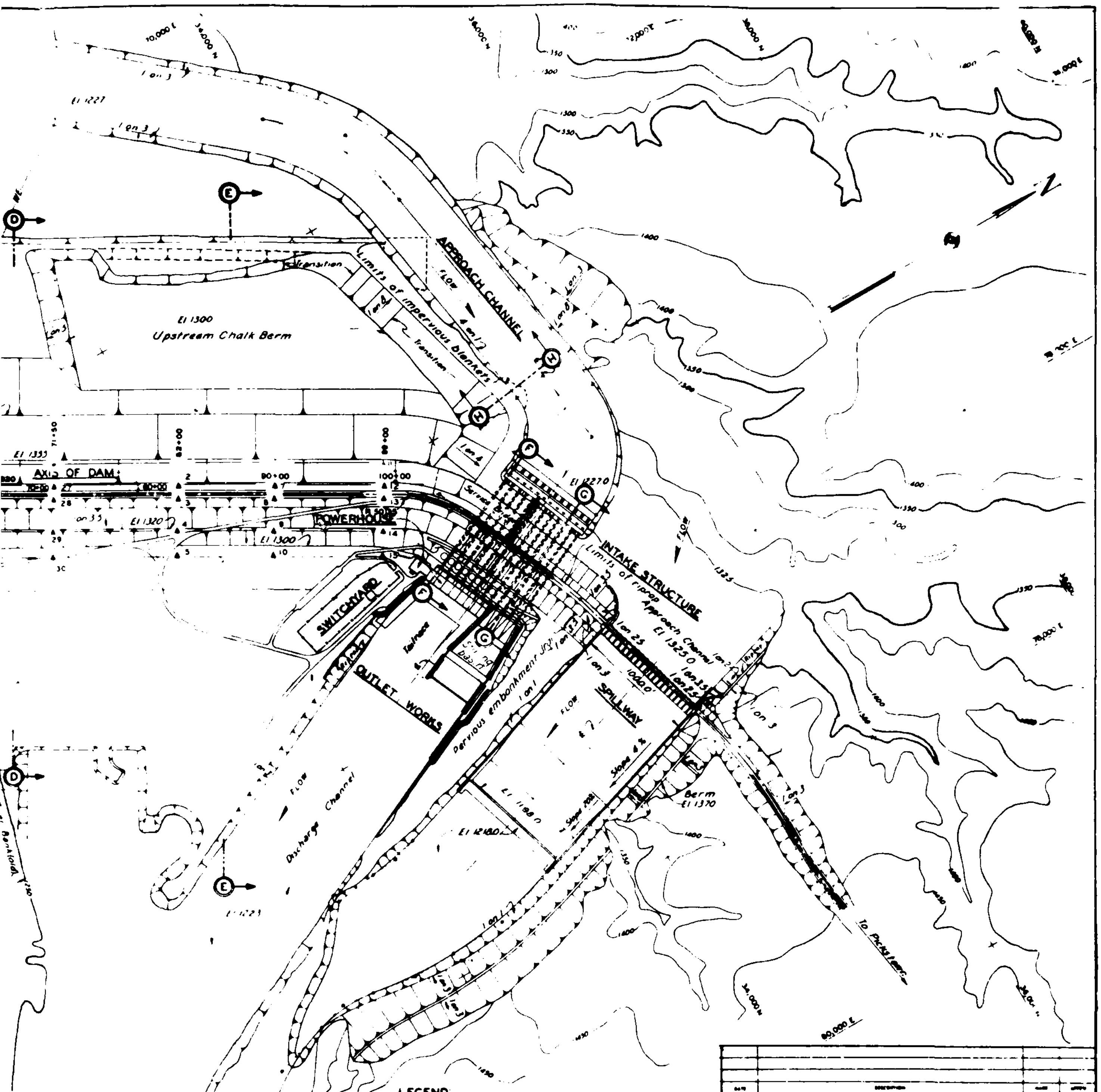
2



TABLE

SETTLEMENT GAGE DATA

Gage Number	Station	Range	Initial Elevation Of Settlement Plate	Length Of Settlement Pipe
31	22+50	4950	1354 82	35 03
32	22+50	5100	1345 72	28 74
33	30+00	4890	1328 63	55 49
34	30+00	5035	1330 56	67 33
35	30+00	5250	1333 65	13 44
36	40+00	4890	1309 16	77 31
37	40+00	5035	1308 59	90 65
38	40+00	5250	1313 39	34 21
16	48+00	4890	1281 347	103 64
17	48+00	5035	1281 123	118 04
18	48+00	5250	1301 238	46 38
39	48+00	5510	1302 27	16 70
19	57+00	4890	1266 285	117 37
20	57+00	5160	1266 131	40 37
22	65+00	4920	1241 985	146 02
23	65+00	5035	1254 06	143 75
24	65+00	5300	1250 32	69 72
25	65+00	5510	1247 99	56 34
27	71+50	4920	1242 27	146 19
28	71+50	5035	1251 60	144 48
29	71+50	5300	1251 07	68 62
30	71+50	5510	1248 22	56 81
2	82+00	4920	1243 112	146 67
3	82+00	5035	1242 813	157 92
4	82+00	5300	1242 899	84 82
5	82+00	5510	1243 784	63 49
7	90+00	4920	1247 416	141 53
8	90+00	5035	1248 106	147 47
9	90+00	5300	1248 375	72 40
10	90+00	5510	1245 859	61 85
12	99+00	4920	1244 702	142 65
13	99+00	5035	1245 758	149 49
14	99+00	5300	1244 727	75 47
15	99+00	5510	1244 027	48 42



LEGEND

▲ Settlement gages

NOTE For location of Sections A A B B C C D D and E E see Plate A4

For location of Sections F F G G and H H, see Plate A.

THIS DRAWING HAS BEEN REDUCED TO THREE-EIGHTHS THE ORIGINAL SCALE.

SCALE 1 INCH = 400 FEET

THIS DRAWING HAS BEEN REDUCED TO THREE EIGHTHS THE ORIGINAL SCALE

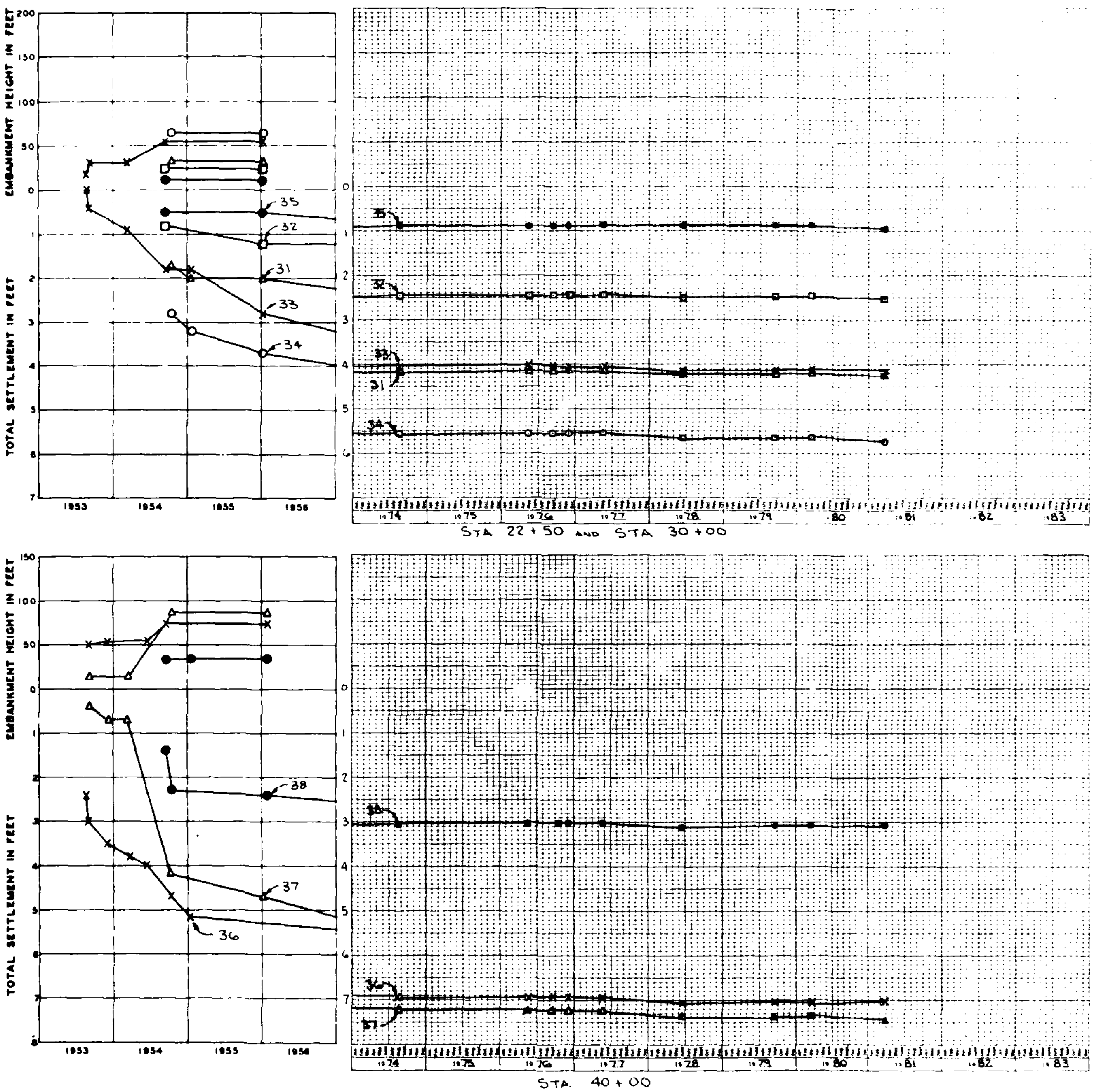
CORPS OF ENGINEERS U S ARMY
OFFICE OF THE DISTRICT ENGINEER
OMAHA DISTRICT
OMAHA NEBRASKA

MISSOURI RIVER

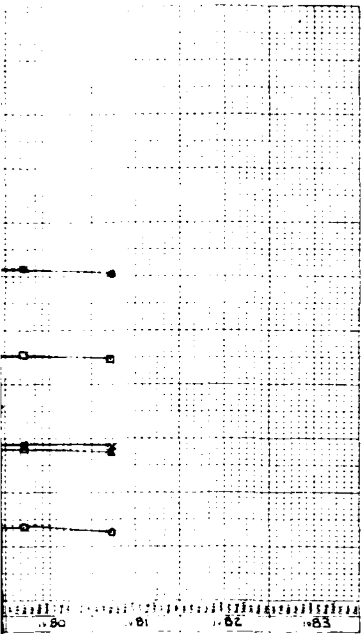
FORT RANDALL DAM - LAKE FRANCIS CASE RESERVOIR & POWERHOUSE

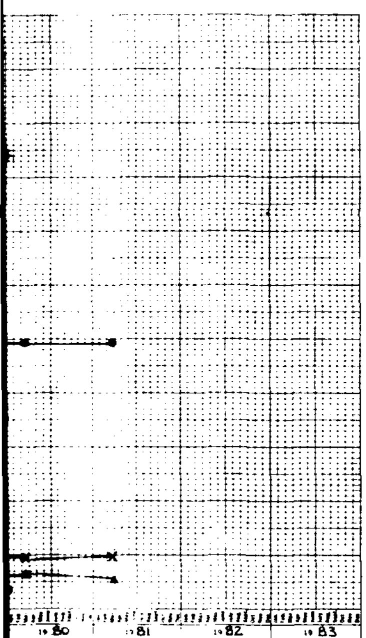
GENERAL PLAN & SETTLEMENT GAGE LOCATIONS

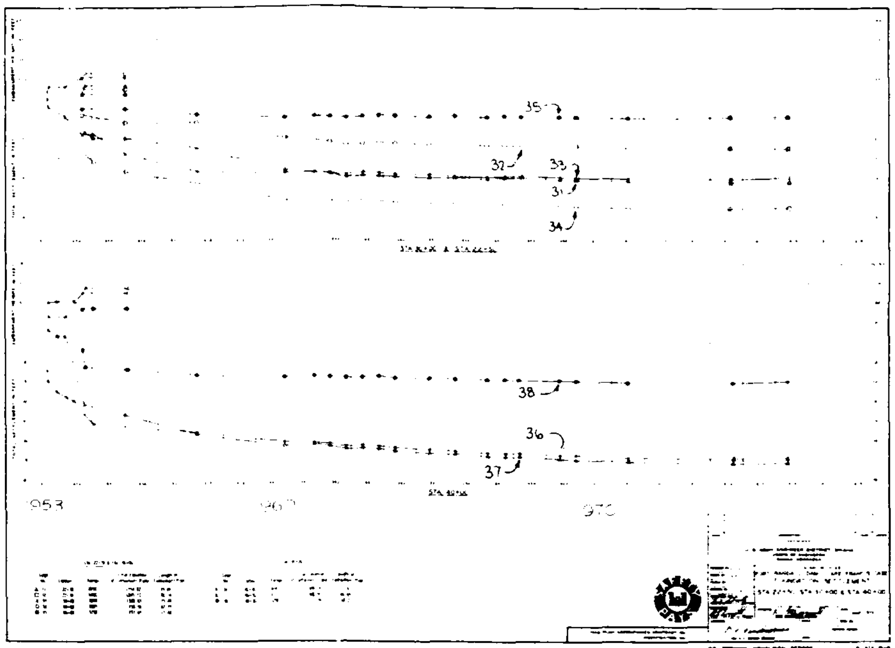
2



SETTLEMENT PLOTS, 1974-1981







SETTLEMENT PLOTS, 1953 - 1976

THIS DRAWING HAS BEEN REDUCED TO THREE-EIGHTHS THE ORIGINAL SCALE.

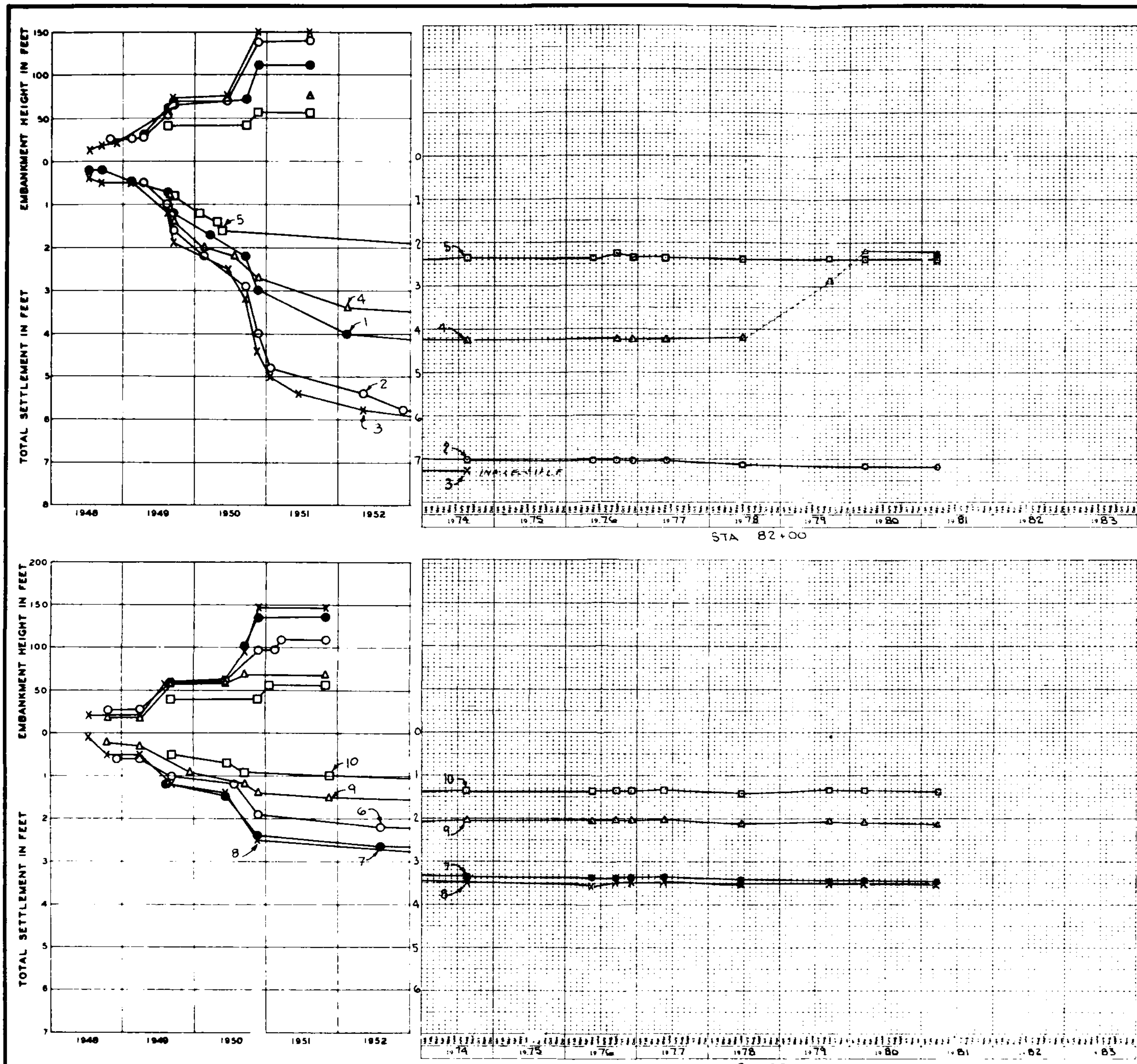
DATE	DESCRIPTION	MADE	APPR'D
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

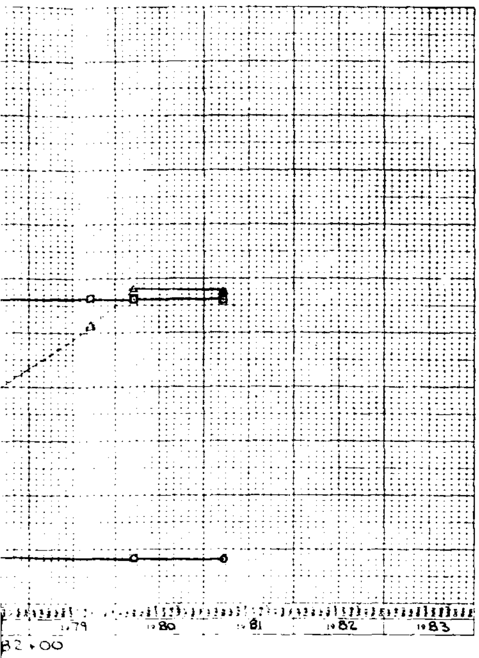
MISSOURI RIVER
FORT RANDALL DAM
FOUNDATION SETTLEMENT
STA 22+50, STA 30+00 & STA 40+00

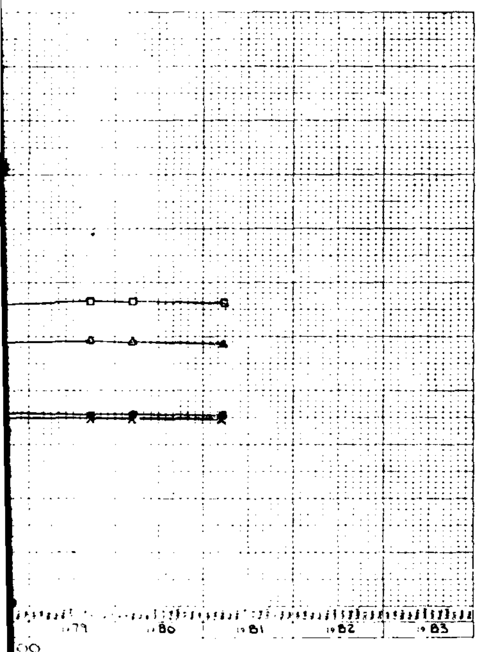
SCALE AS SHOWN

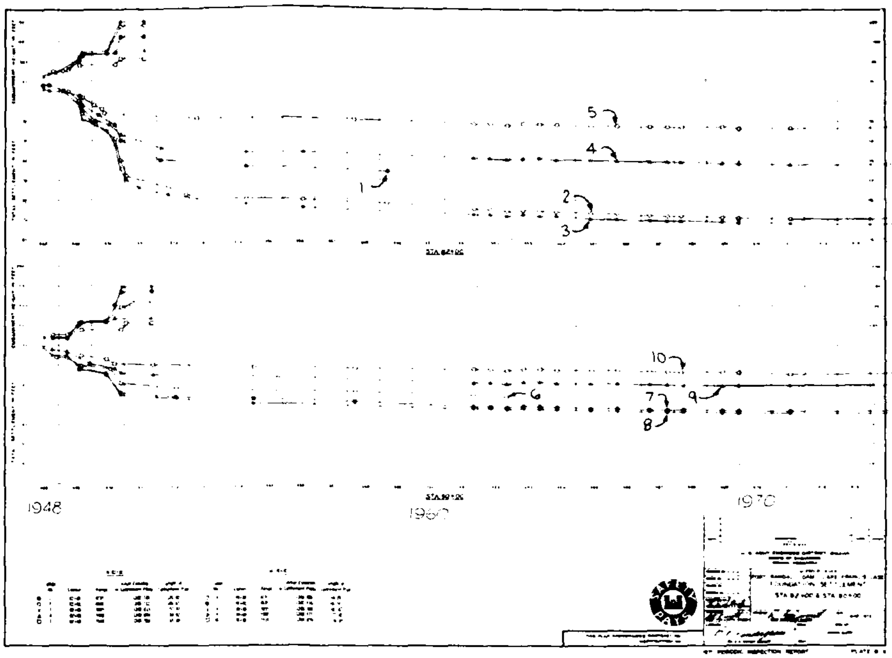
THIS PLAN ACCOMPANIES CONTRACT NO.
MODIFICATION NO.



SETTLEMENT PLOTS, 1974-1981







SETTLEMENT PLOTS, 1948-1973

REVISIONS

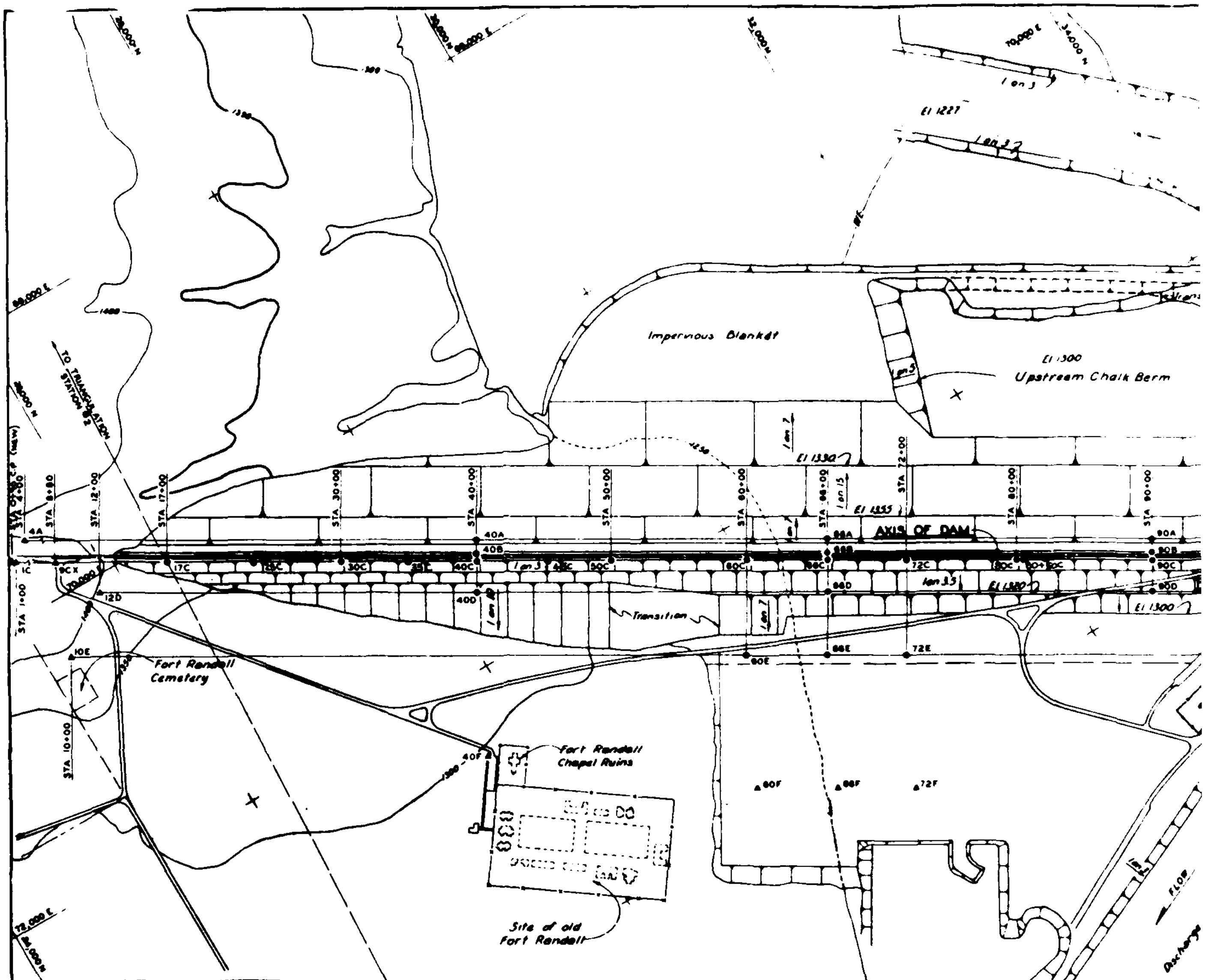
U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

MISSOURI RIVER
FORT RANDALL DAM
FOUNDATION SETTLEMENT
STA 82+00 & STA 90+00

SAFETY PAYS

THIS PLAN ACCOMPANIES CONTRACT No.
MODIFICATION No.

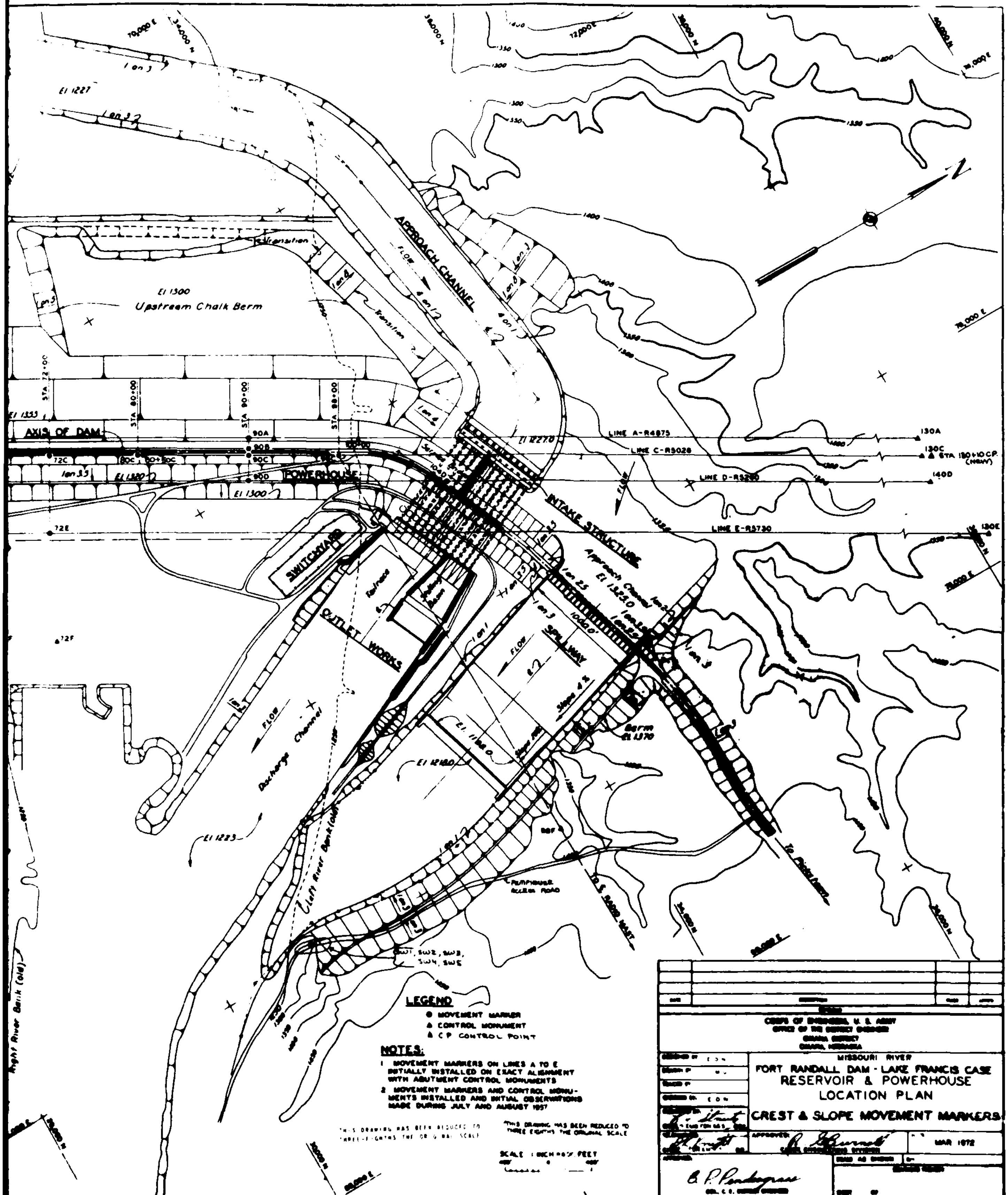
2



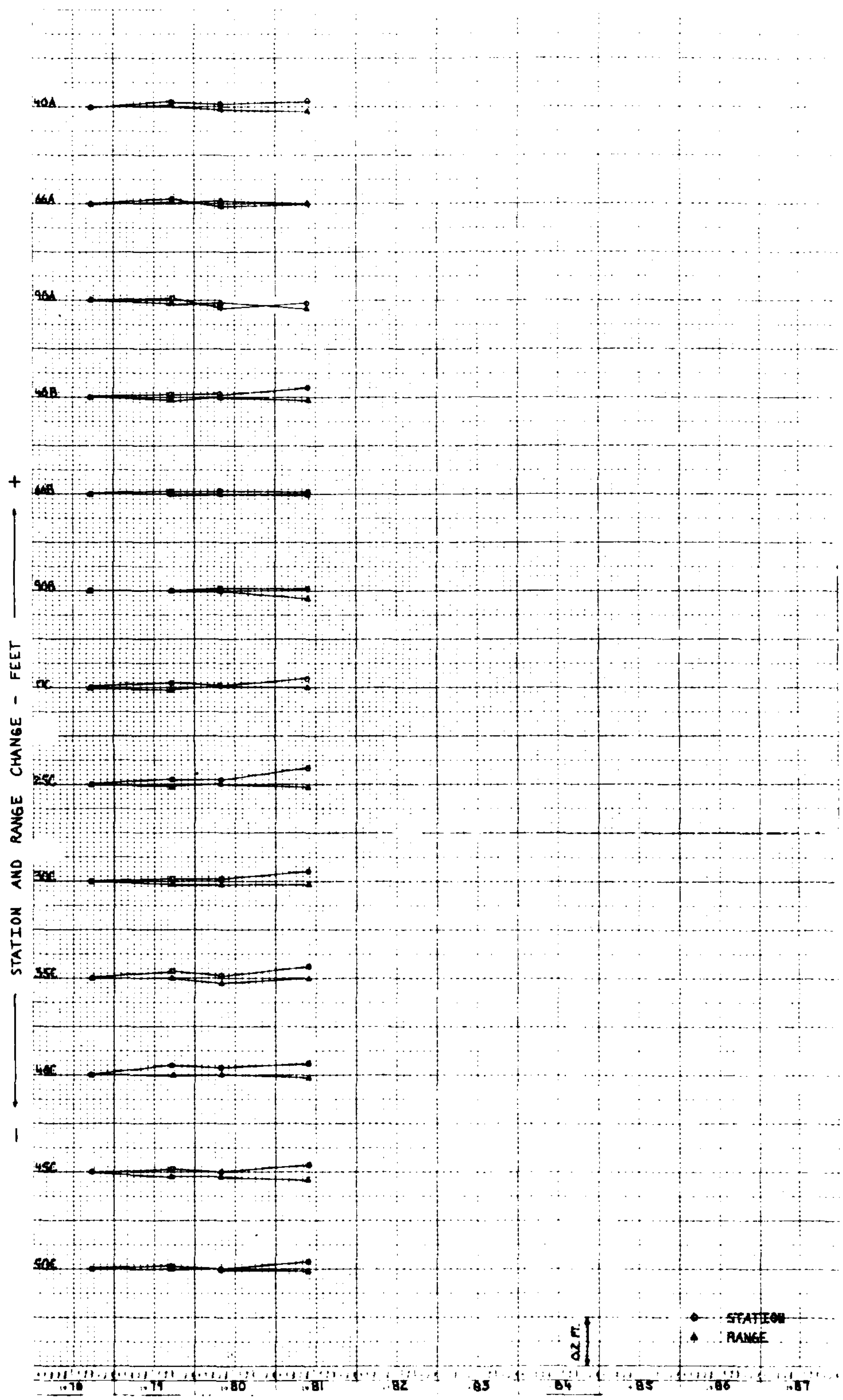
INITIAL ELEVATIONS OF MARKERS		
MARKER NO.	INITIAL ELEV	REMARKS
40A	1378.415	REPLACED WITH FROST FREE MOVEMENT MARKERS WINTER OF 1980-1981
66A	1378.411	
90A	1378.240	
40B	1396.240	
66B	1394.914	
90B	1394.187	
9C	1424.221	
17C	1397.654	
30C	1396.166	
40C	1395.776	
50C	1395.826	
60C	1394.599	
66C	1395.115	
72C	1395.035	
80C	1395.511	
90C	1394.790	
98C	1396.474	
40D	1342.236	
66D	1325.667	
90D	1324.616	
104D	1304.962	
60E	1301.228	
66E	1301.125	
72E	1300.046	
25C	1395.925	
35C	1396.034	
45C	1394.246	
91+30C	1394.710	

INITIAL DISTANCES BETWEEN MARKERS (C-LINE)	
MARKER NO	DISTANCE IN FEET
9CX	
	820.868
17C	
	1323.738
30C	
	999.724
40C	
	999.792
50C	
	999.756
60C	
	599.870
66C	
	599.982
72C	
	799.912
80C	
	1000.190
90C	
	800.136
98C	

1


APPROACH CHANNEL
FLOW
El 1300
Upstream Chalk Berm
AXIS OF DAM
POWERHOUSE
INTAKE STRUCTURE
SWITCHYARD
OUTLET WORKS
SPILLWAY
Discharge Channel
Left River Bank (old)
Right River Bank (old)
Berm El 1370
LINE A-R4875
LINE C-R5028
LINE D-R5200
LINE E-R5730
130A
130C
STA 130+10 C.P. (NEW)
140D
130E
72C
72E
72F
90A
90B
90C
90D
STA 72+00
STA 80+00
STA 90+00
STA 98+00
SW1, SW2, SW3, SW4, SW5
Pumphouse Access Road
LEGEND
MOVEMENT MARKER
CONTROL MONUMENT
C P CONTROL POINT
NOTES:
1 MOVEMENT MARKERS ON LINES A TO E INITIALLY INSTALLED ON EXACT ALIGNMENT WITH ABUTMENT CONTROL MONUMENTS
2 MOVEMENT MARKERS AND CONTROL MONUMENTS INSTALLED AND INITIAL OBSERVATIONS MADE DURING JULY AND AUGUST 1957
THIS DRAWING HAS BEEN REDUCED TO THREE EIGHTHS THE ORIGINAL SCALE
MISSOURI RIVER
FORT RANDALL DAM - LAKE FRANCIS CASE
RESERVOIR & POWERHOUSE
LOCATION PLAN
CREST & SLOPE MOVEMENT MARKERS
MAR 1972

2


40A
66A
90A
40B
44B
50B
0C
25C
30C
35C
40C
45C
50C
+ STATION AND RANGE CHANGE - FEET -
0.2 FT.
STATION
RANGE
78
79
80
81
82
83
84
85
86
87

1

THIS DRAWING HAS BEEN REDUCED TO
THREE-EIGHTHS THE ORIGINAL SCALE

FOR LOCATION PLAN SEE PLATE A56

REVISIONS

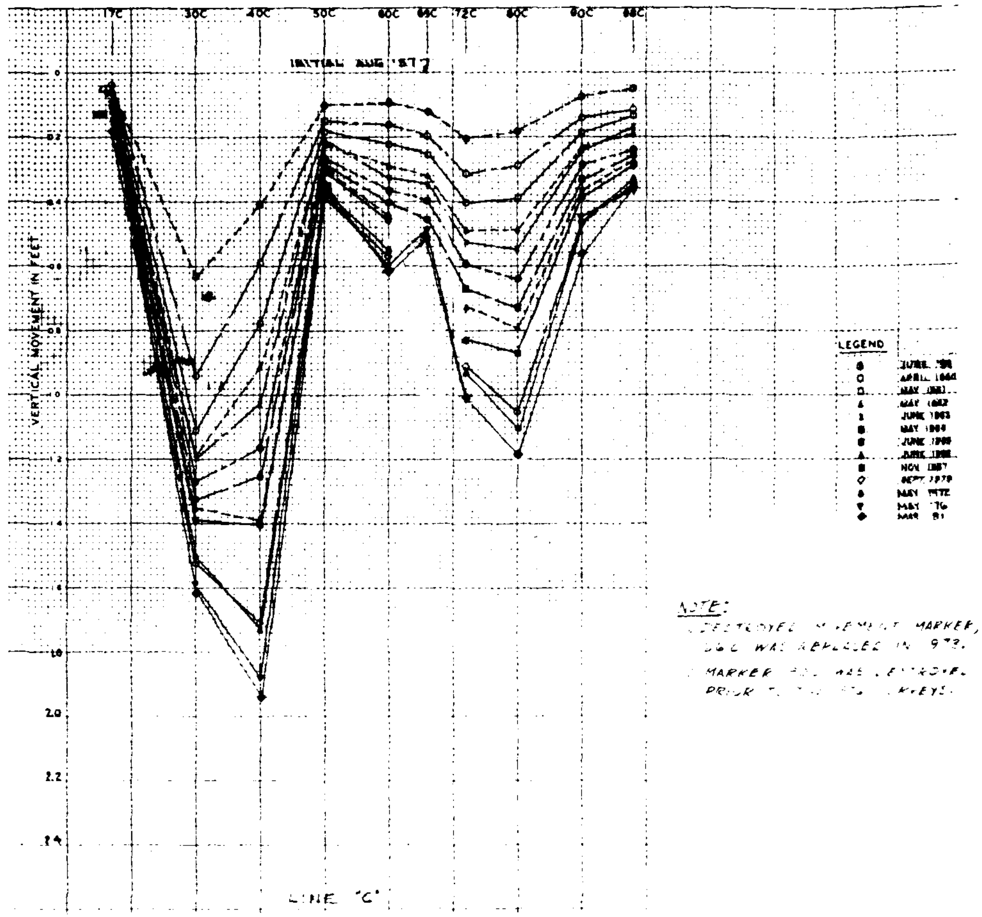
U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

MISSOURI RIVER

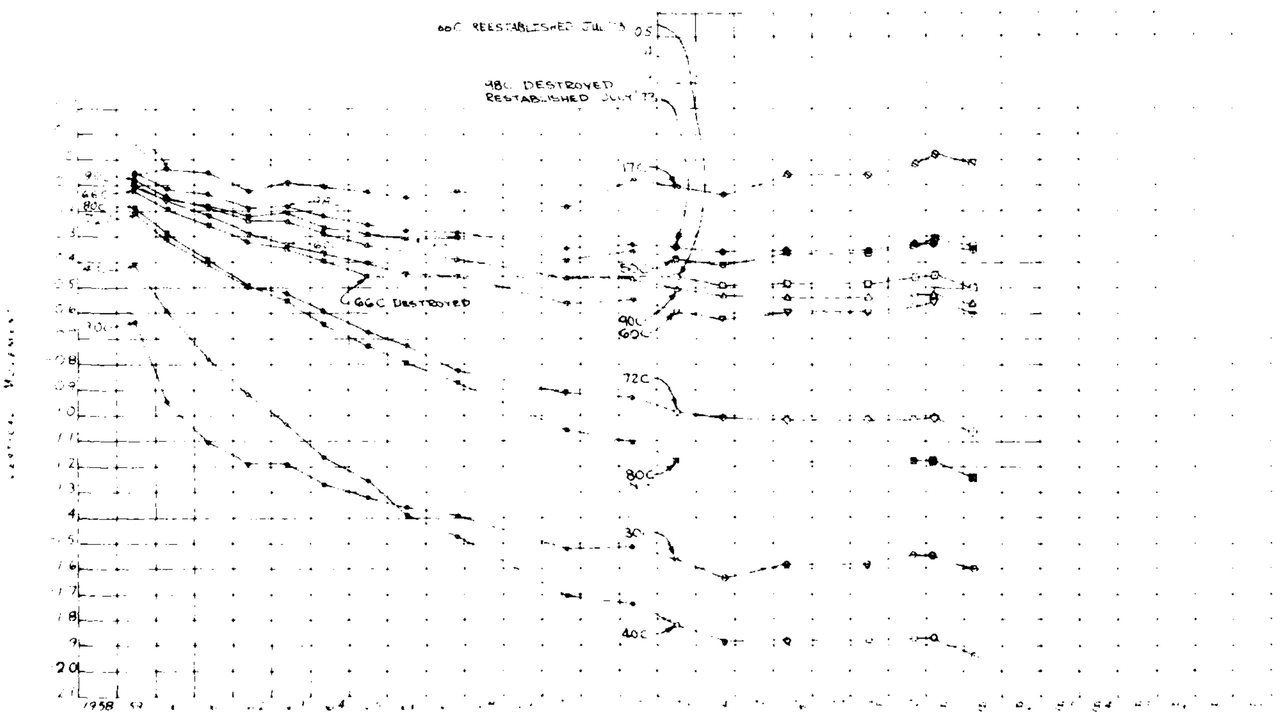
FORT RANDALL DAM
CREST AND SLOPE MOVEMENT MARKERS
HORIZONTAL MOVEMENT

SCALE AS SHOWN

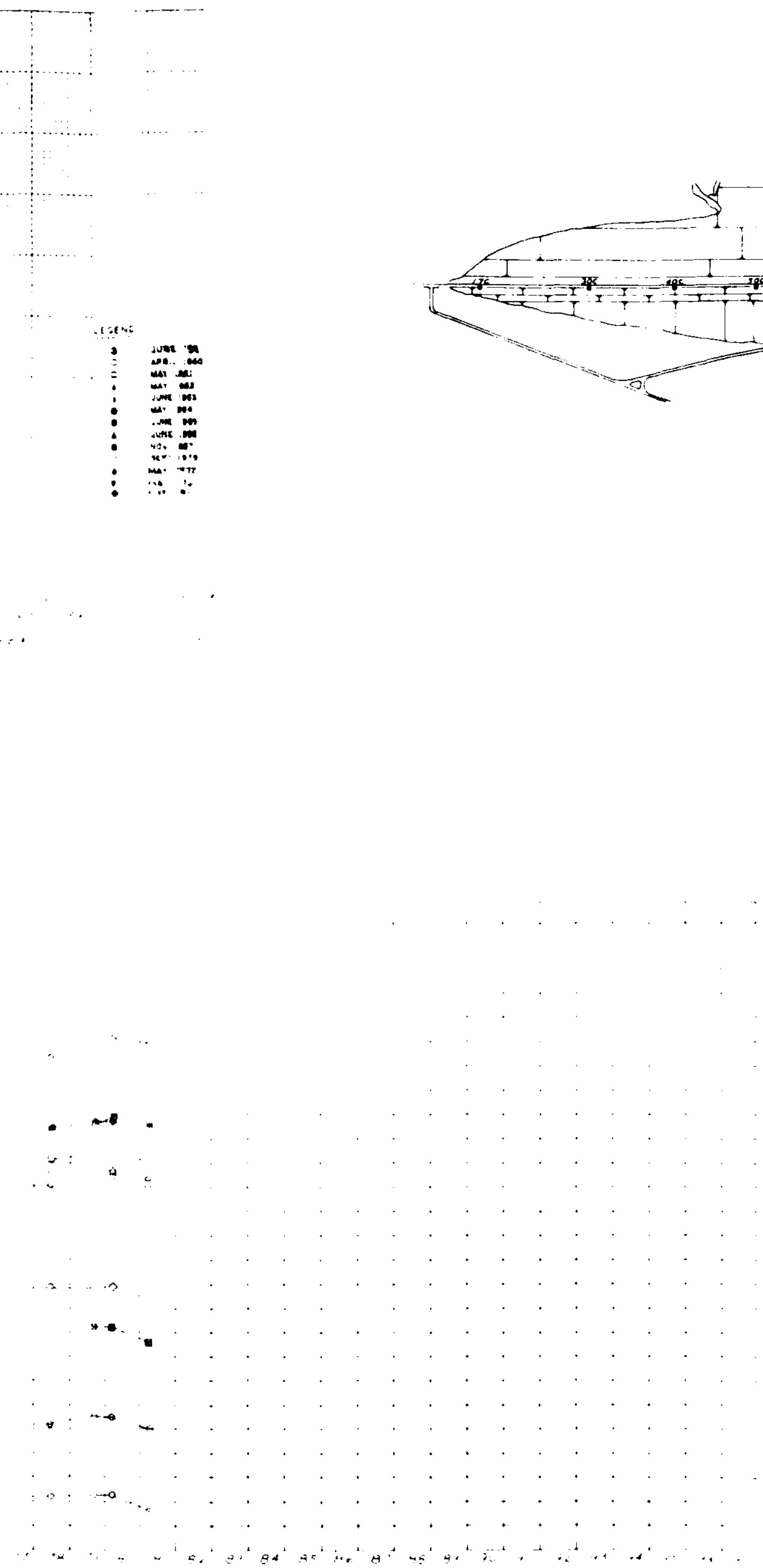
THIS PLAN ACCOMPANIES CONTRACT No.
MODIFICATION No.

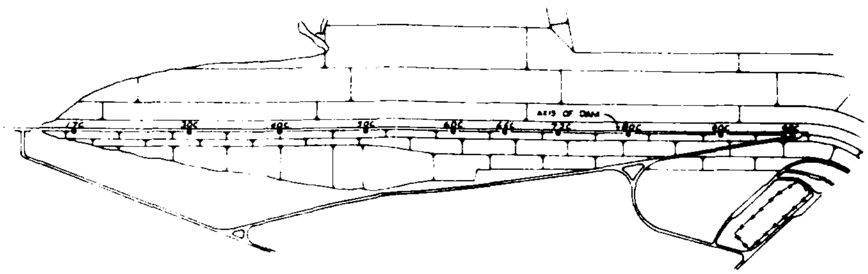


INITIAL AUG '57
VERTICAL MOVEMENT IN FEET
LEGEND
JUNE '58
APRIL 1960
MAY 1961
MAY 1962
JUNE 1963
MAY 1964
JUNE 1965
JUNE 1966
NOV 1967
SEPT 1970
MAY 1972
MAY '76
MAY '81
LINE "C"



66C REESTABLISHED JULY '73
48C DESTROYED
RESTABLISHED JULY '73
66C DESTROYED
17C
90C
66C
72C
80C
30C
40C





KEY PLAN

THIS DRAWING HAS BEEN REDUCED TO THREE-FOURTHS THE ORIGINAL SCALE

DATE	DESCRIPTION	MADE	APPR'D
	REVISIONS		

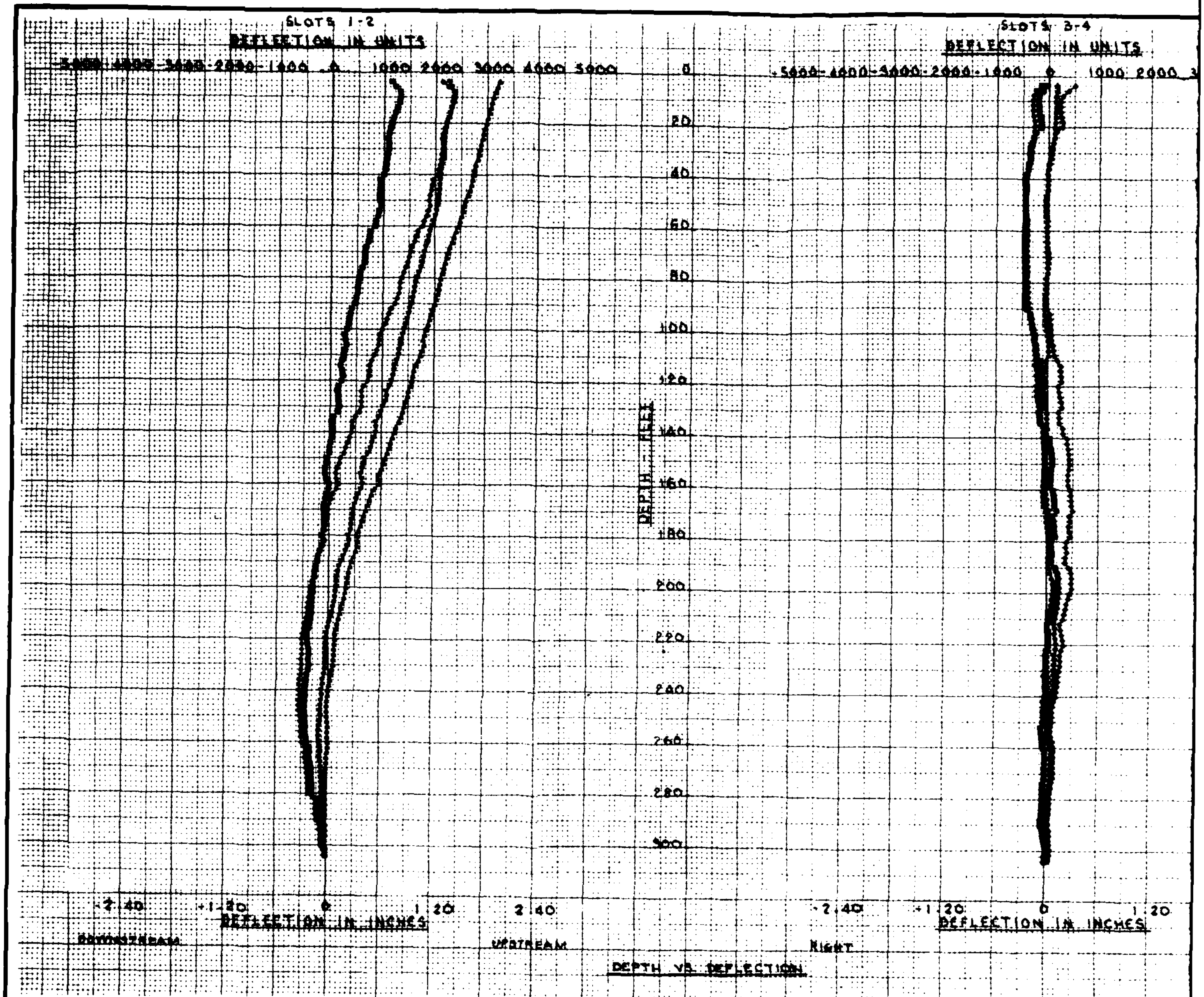
U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

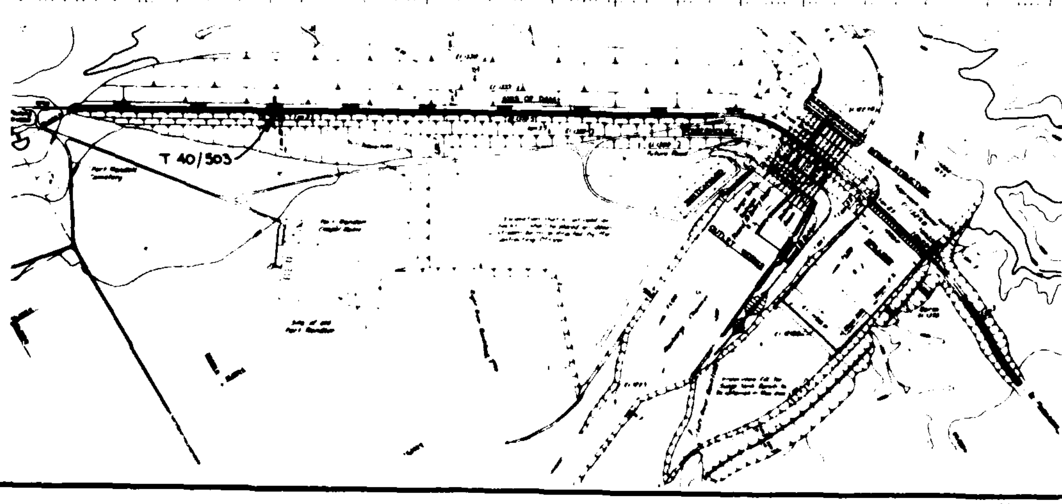
MISSOURI RIVER
FORT RANDALL DAM
SLOPE AND CREST MOVEMENT MARKERS
VERTICAL MOVEMENT
LINE 'C'

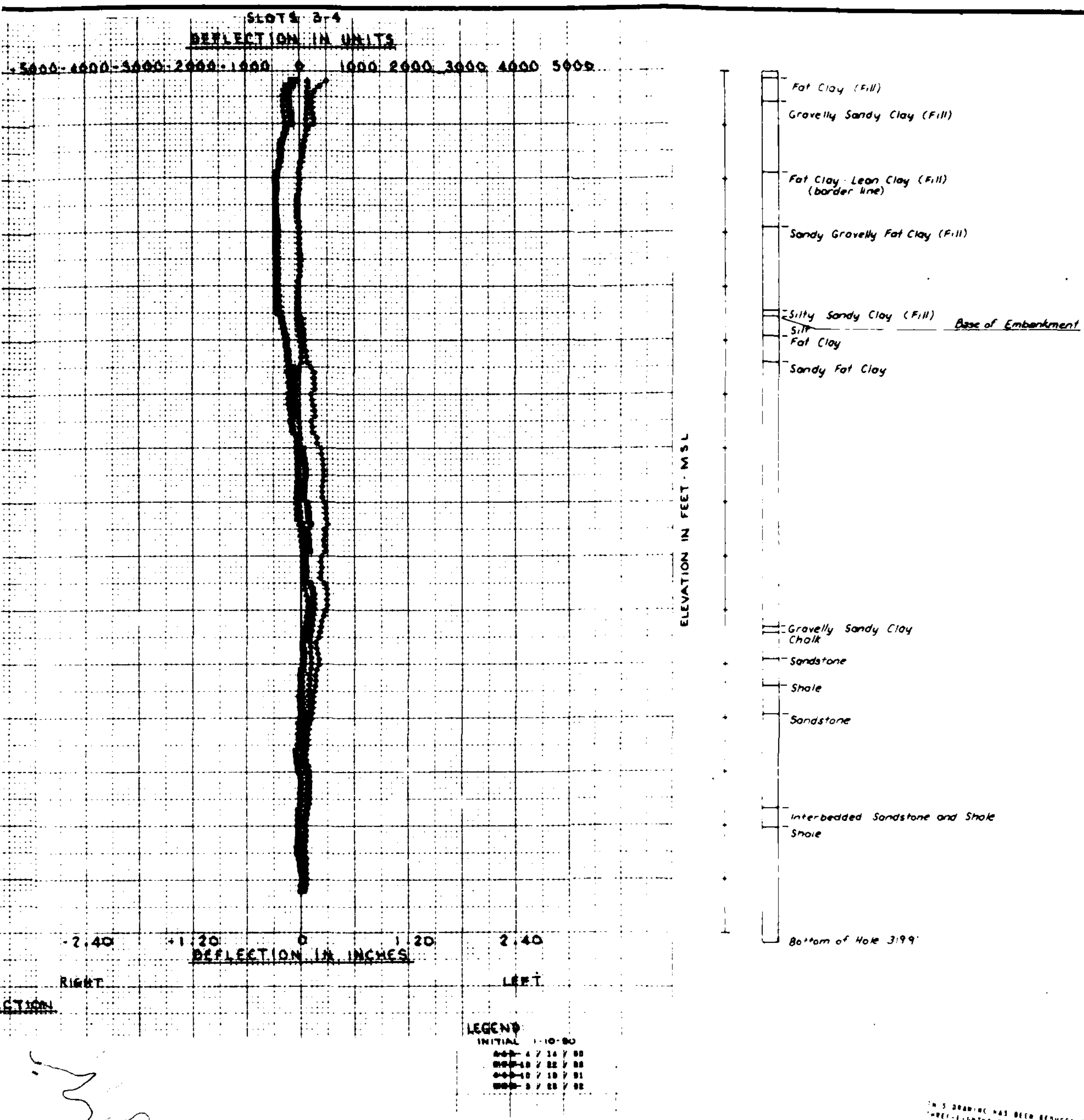
DESIGNED BY:
DRAWN BY:
CHECKED BY:
SUBMITTED BY:
CHIEF SECTION
RECOMMENDED:
CHIEF BRANCH
APPROVED:
APPROVED CHIEF ENGINEERING DIVISION
COL. C. E. DISTRICT ENGINEER
DATE
SCALE AS SHOWN
SPEC NO.
DRAWING NUMBER
SHEET

SAFETY PAYS

THIS PLAN ACCOMPANIES CONTRACT NO.
MODIFICATION NO.


SLOTS 1-2
DEFLECTION IN UNITS
SLOTS 3-4
DEFLECTION IN UNITS
DEPTH - FEET
DEFLECTION IN INCHES
DOWNSTREAM
UPSTREAM
RIGHT
DEFLECTION IN INCHES
DEPTH VS DEFLECTION


T 40/503



THIS DRAWING HAS BEEN REDUCED TO THREE-EIGHTHS THE ORIGINAL SCALE

REVISIONS

U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

MISSOURI RIVER
FORT RANDALL DAM
TILTMETER OBSERVATIONS
T 40/503
EMB STA 40+00 RANGE 5030

SCALE: AS SHOWN

THIS PLAN ACCOMPANIES CONTRACT NO. DACA45 MODIFICATION NO.

$$ – THINK VALUE ENGINEERING – $$

APPENDIX B
PHOTOS

PHOTO NO. 1: Aerial view of Fort Randall Dam. June 1974

PHOTO NO. 2: Scarifying surface prior to placement of fill. Earthwork Stage II. 20 May '49

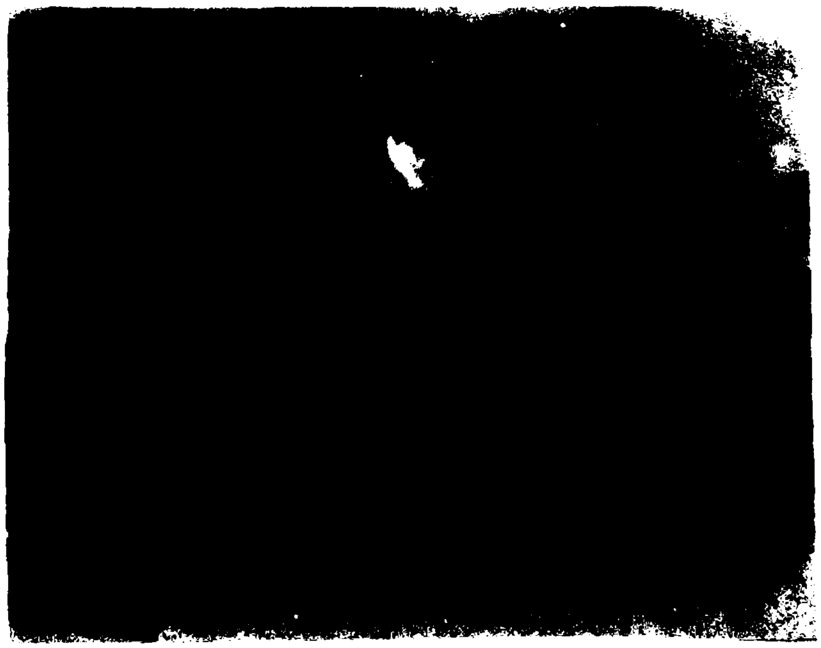


PHOTO NO. 3: Blading operation in embankment construction. Initial earthwork.

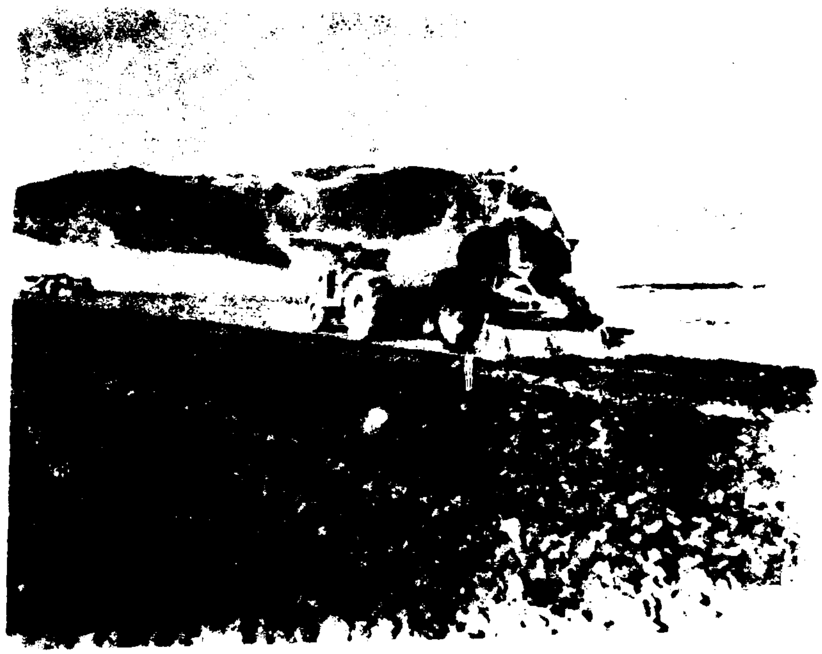


PHOTO NO. 4: Watering operation in embankment construction. Earthwork Stage II. 20 May '49

PHOTO NO. 5: Excavation of right bank cut-off trench, upstream end, looking towards river. 6 Nov '50

PLATE B-3

PHOTO NO. 6: Sheepsfoot tamping roller compacting impervious fill. Earthwork Stage III. 10 Oct '50

PHOTO NO. 7: Construction of intake structure and embankment in foreground and excavation for spillway weir in background, looking toward the left abutment. 11 May '51

PHOTO NO. 8: Dredging in chalk spoil area downstream of embankment for hydraulic filling to effect river closure. Earthwork Stage III. 17 Jul '52

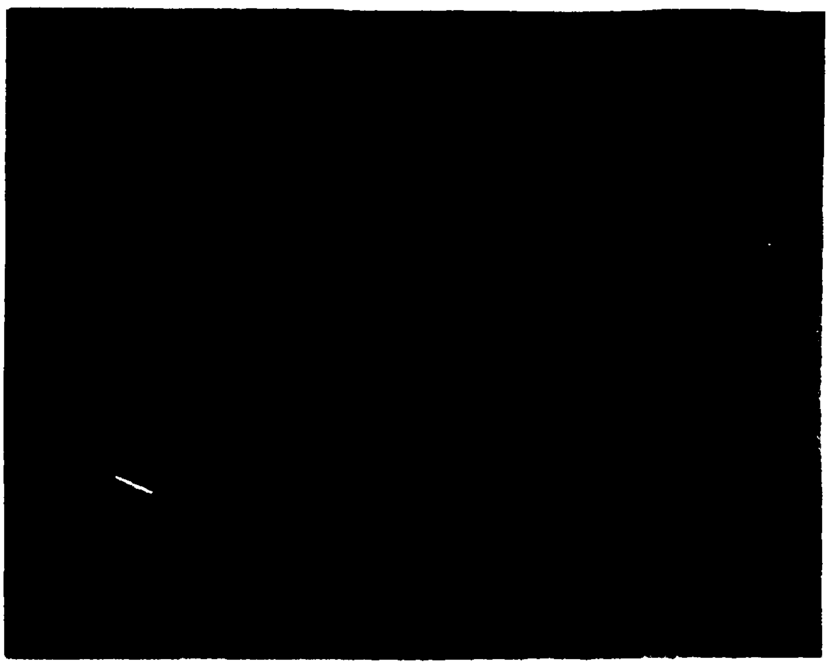


PHOTO NO. 9: Final stage of river closure with dredged chalk fill. Water level at about El. 1242. 20 Jul '52

PHOTO NO. 10: Dumped chalk fill over the dredged chalk weir crest. Earthwork Stage III. 23 Jul '52

PHOTO NO. 11: Placement of pervious dike at location of upstream toe of impervious blanket. Earthwork Stage III. 1 Aug '52

PHOTO NO. 12: Embankment material being placed on top of dredged hydraulic fill at location of upstream embankment toe. Earthwork Stage III. 1 Aug'52

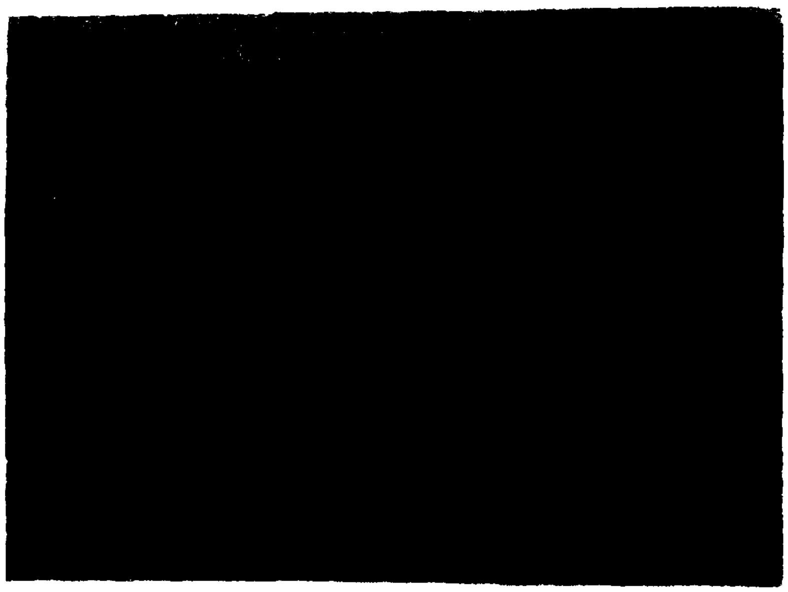


PHOTO NO. 13: Same area as shown in Photo No. 12. Pictured are double cat-dozer and towed double drum sheepsfoot roller. 1 Aug '52

PHOTO NO. 14: Hydraulic filling of embankment foundation in closure section. Earthwork Stage III. 15 Sep '52.

PHOTO NO. 15: Aerial view of project looking downstream. 15 Sep '52

PHOTO NO. 16: Embankment construction in closure area looking towards the left abutment. Fill at approx. El. 1246. Earthwork Stage III. 14 Aug '52.

PHOTO NO. 17: Upstream portion of embankment, looking towards the right abutment. Fill is at approx. El. 1288. Earthwork Stage III. 29 Sep '52

PHOTO NO. 18: Embankment construction in closure area to approx. El. 1325. Embankment in foreground is at crest level, El. 1395. Earthwork Stage III.
(17 Oct '52)

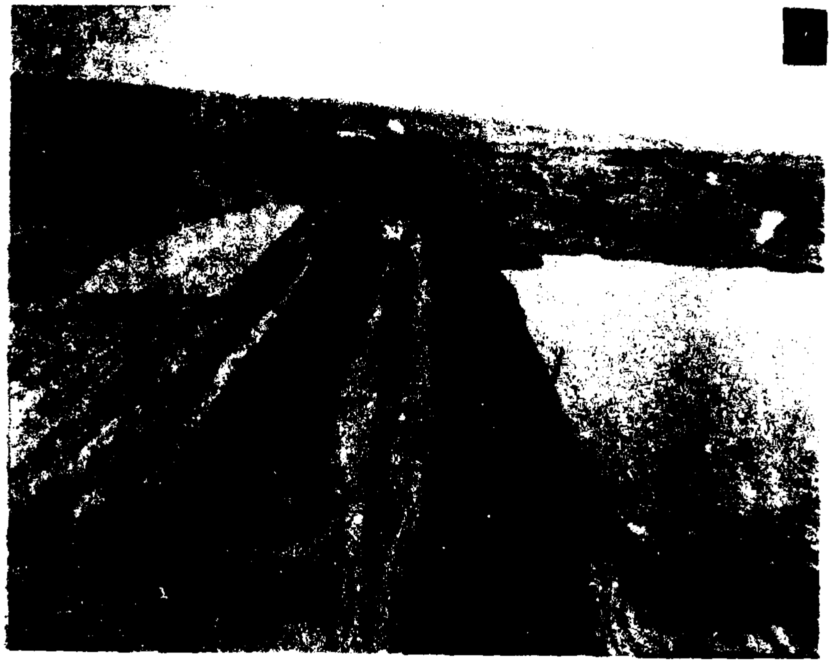


PHOTO NO. 19: View of embankment construction looking towards right abutment, showing placement of upstream chalk berm. Earthwork Stage IV. 4 Aug '53

PHOTO NO. 20: Construction of upstream chalk berm looking SW towards right abutment. 4 Aug '53

PHOTO NO. 21: Aerial view of construction during Earthwork Stage V. 16 Nov '53

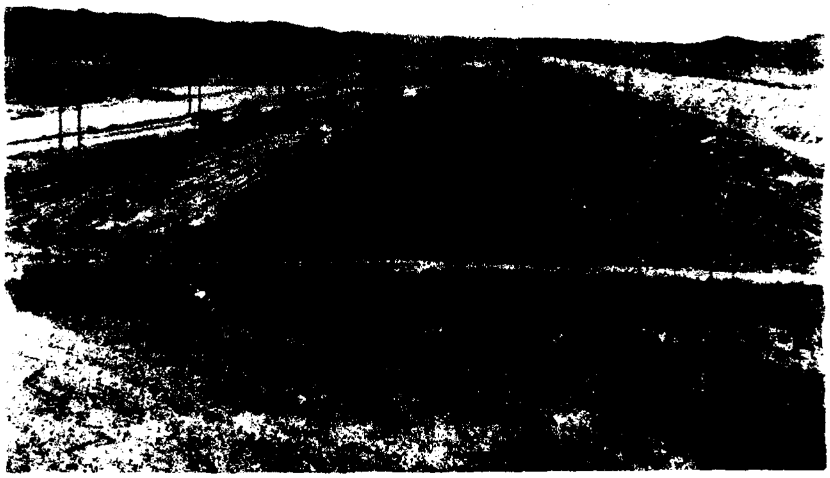


PHOTO NO. 22: View of embankment construction looking along dam axis towards the right abutment. Earthwork Stage V. 25 May '54

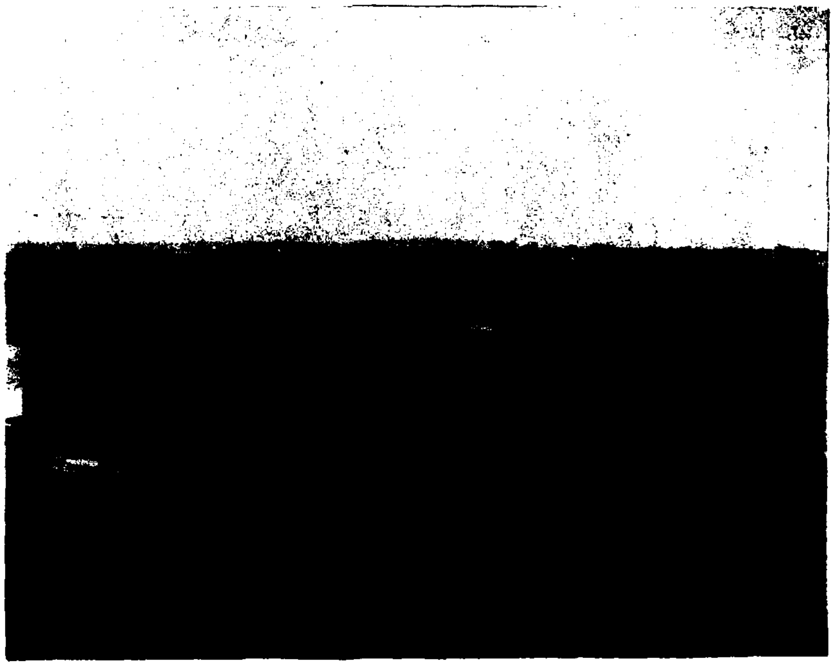


PHOTO NO. 23: Fill placement between west spillway abutment and previously completed embankment. Top of fill is approx. El. 1360. 25 Jun '54.

PHOTO NO. 24: Aerial view of completed embankment, looking from right abutment. 30 Jun '55

PHOTO NO. 25: Aerial view of project, looking SW. 29 Mar '55

END

DATE
FILMED

10 83

DTIC